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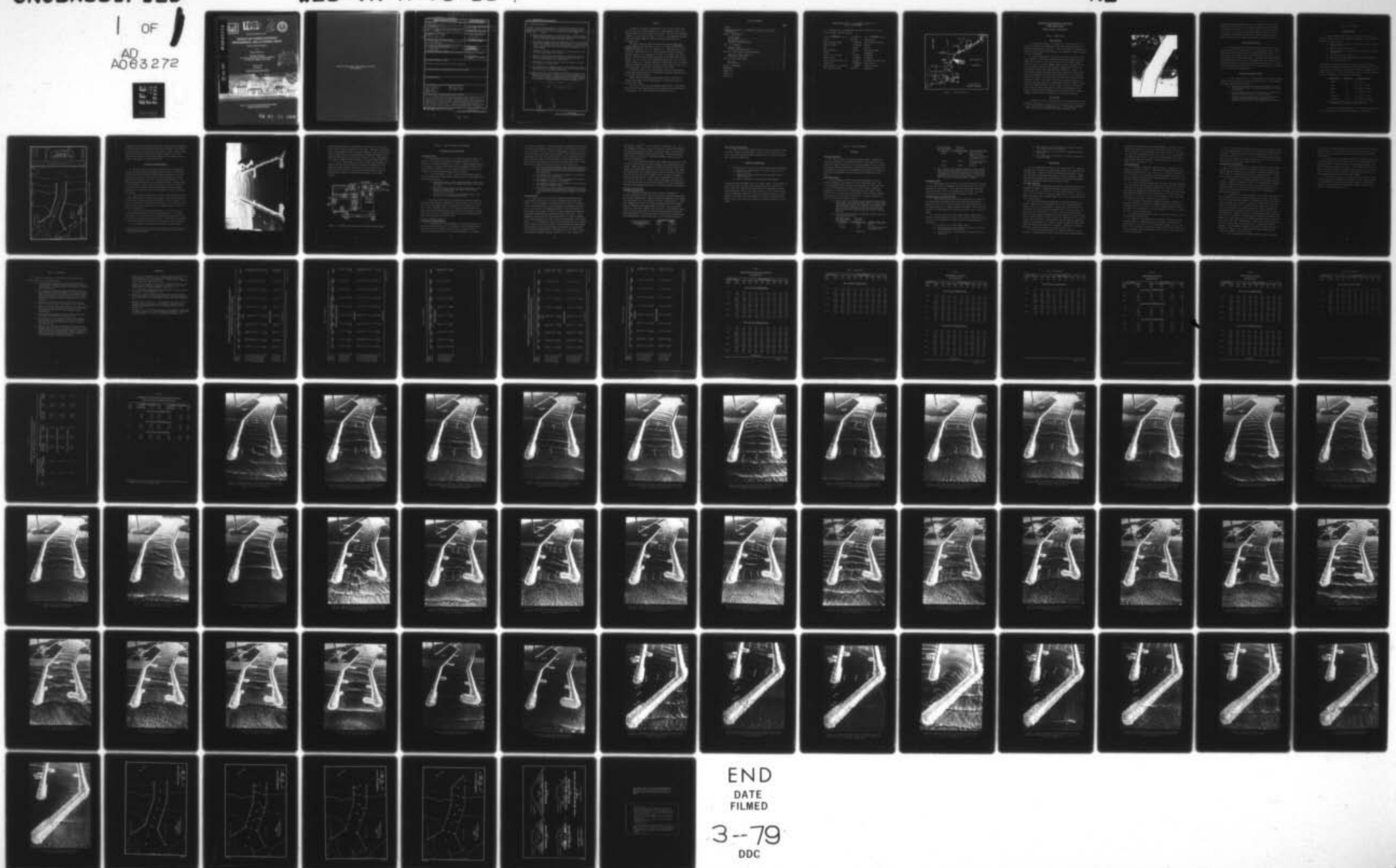

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TECHNICAL REPORT H-78-18

DESIGN FOR HARBOR ENTRANCE IMPROVEMENTS, WELLS HARBOR, MAINE

Hydraulic Model Investigation

by

Robert R. Bottin, Jr.

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A 1:50-scale (undistorted) hydraulic model of Wells Harbor, Maine, which included the entrance to the harbor, approximately 900 ft and 1100 ft of shoreline to the south and north of the harbor, respectively, and sufficient offshore area in the Gulf of Maine to permit generation of the required test waves, was used to investigate the effects of proposed improvements at the harbor entrance on wave action. Proposed improvement plans consisted of the installation of stone spur dikes in the jettied entrance and a breakwater

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20. ABSTRACT (Continued)

attached to the existing north jetty. A 20-ft-long wave generator, a model circulation system, and an automated data acquisition and control system ~~(ADAGS)~~ were ~~utilized~~ used in model operation. It was concluded from model tests showed that results that:

a. During periods of moderate to large wave attack, the existing harbor entrance experiences hazardous navigation conditions due to breaking waves and interaction of waves with tidal currents.

b. The originally proposed spur-dike configuration (plan 1) will slightly increase wave heights in the outer entrance (gage 1), will have little effect on wave heights between the outer entrance and the bend in the jetties, and will reduce wave heights through the remainder of the jettied entrance.

c. Changing the spur-dike cross sections at the entrance (plan 2) will not reduce wave heights at that location.

d. Removal of the first spur dike attached to the south jetty and the third spur dike attached to the north jetty (approaching from the sea) will not compromise design effectiveness.

e. Shoaling may occur between the spur dikes, particularly the longer ones attached to the south jetty.

f. The installation of a breakwater (plan 3) will substantially improve wave conditions in the jettied entrance over those for existing conditions or any of the spur-dike plans tested.

g. Whether tidal velocities following installation of the proposed spur dikes will be sufficient to maintain a self-scouring channel at described navigable depths is beyond the scope of this ~~model~~ investigation. However, qualitative indications are that the spur dikes will be beneficial in reducing maintenance dredging requirements.

several spur-dike, jetty and breakwater configurations were examined for their effect on wave conditions.

PREFACE

A request for a model investigation of Wells Harbor, Maine, was initiated by the Division Engineer, U. S. Army Engineer Division, New England (NED), in a letter dated 14 June 1977. The study was authorized by NED on 13 July 1977, and funds for the U. S. Army Engineer Waterways Experiment Station (WES) to conduct the study were authorized on 15 August 1977.

The model study was conducted during the period October 1977-January 1978 under the direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory; Mr. F. A. Herrmann, Jr., Assistant Chief of the Hydraulics Laboratory; Dr. R. W. Whalin, Chief of the Wave Dynamics Division; and Mr. C. E. Chatham, Jr., Chief of the Harbor Wave Action Branch. Testing was performed by personnel of the Harbor Wave Action Branch: Mr. H. F. Acuff, Civil Engineering Technician, with the assistance of Messrs. R. E. Ankeny, Electronics Technician, and K. A. Turner, Computer Specialist, under the supervision of Mr. Robert R. Bottin, Jr., Project Engineer. This report was prepared by Mr. Bottin.

During the course of the investigation, liaison was maintained between NED and WES by means of conferences, telephone communications, and monthly progress reports.

Messrs. Carl Hard and Frank Notardonato, NED, visited WES to observe model operations and participate in conferences during the course of the study.

COL John L. Cannon, CE, was Commander and Director of WES during the conduct of this investigation and the preparation and publication of this report. Mr. Fred R. Brown was Technical Director.

TABLE OF CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) as follows:

Multiply	By	To Obtain
acres	4046.856	square metres
cubic feet per second	0.02831685	cubic metres per second
degrees (angle)	0.01745329	radians
feet	0.3048	metres
feet per second	0.3048	metres per second
inches	25.4	millimetres
miles (U. S. statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres
square miles (U. S. statute)	2.589988	square kilometres
tons (2000 lb, mass)	907.1847	kilograms

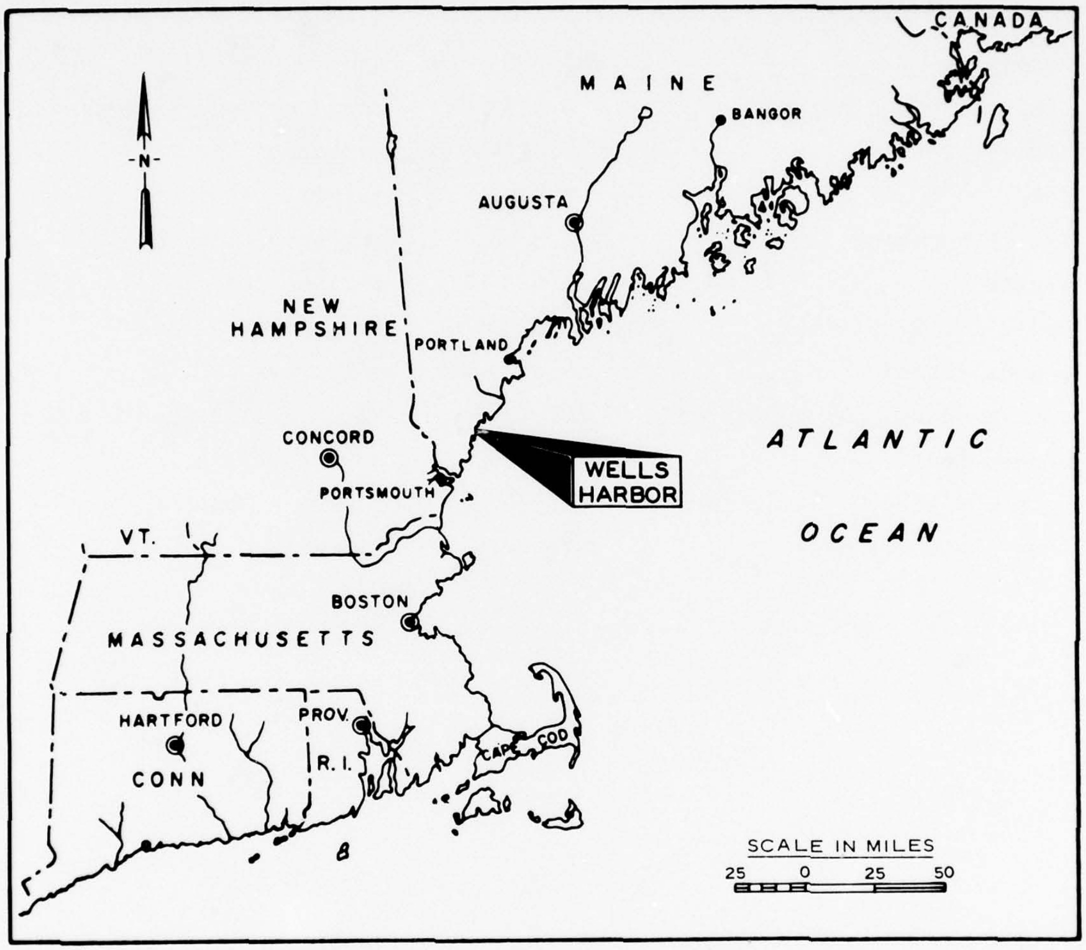

Figure 1. Project location

DESIGN FOR HARBOR ENTRANCE IMPROVEMENTS

WELLS HARBOR, MAINE

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Wells Harbor, Maine, is a small inlet located in the town of Wells at the mouth of the Webhannet River about 20 miles* northeast of Portsmouth Harbor, Maine (Figure 1). Wells is primarily a summer resort area for small pleasure boats; however, some commercial lobster business has been established.

2. Initial improvements at Wells Harbor were authorized in 1961 and consisted of an 8-ft-deep, 100-ft-wide entrance channel extending about 2500 ft from the -8 ft contour in the Gulf of Maine through a jettied entrance to an inner channel where the depth decreased to 6 ft. The 6-ft deep, 150-ft-wide inner channel extended about 500 ft to a 450-ft-wide, 1100-ft-long anchorage basin comprising 7.4 acres. An 840-ft-long north jetty extended from Drakes Island and a 940-ft-long south jetty extended from Wells Beach, resulting in a 400-ft opening between the jetties. Construction of the north and south jetties was completed in 1962. After jetty construction, excessive shoaling occurred in the harbor entrance, compromising the project design. Authorization to extend the north jetty by 1225 ft and the south jetty by 1300 ft was granted in 1965 and construction was completed in 1967. Figure 2 shows an aerial photograph of the entrance to Wells Harbor in September 1975 and Plate 1 shows details of existing conditions.

The Problem

3. Extensions to the north and south jetties were unsuccessful in

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* A table of factors for converting U. S. customary units of measurement to metric (SI) is presented on page 3. All dimensions in this report are given in prototype units unless otherwise noted.

Figure 2. Aerial view of Wells Harbor entrance

eliminating shoaling problems and due to a continued influx of sand through the inlet entrance, the entrance channel does not maintain a self-scouring depth equal to the project depth and frequent dredging is required. In addition, the entrance channel affords little wave protection to small boats navigating the channel during periods of severe storms, which are commonly experienced in the Gulf of Maine.

The Proposed Solution

4. The proposed solution consists of the installation of stone spur dikes in the jettied entrance so that the entrance width would be reduced and project depths would be maintained by tidal flows.[1] The spur dikes in the entrance would act as wave diffraction gaps and partially dissipate incoming wave energy.[1] However, since this geometry, superimposed upon tidal currents, is complex, some wave reflection and interference may occur resulting in hazardous wave conditions. If this occurs, consideration would be given to the installation of a breakwater for more positive wave suppression.

Purpose of the Model Study

5. At the request of the U. S. Army Engineer Division, New England (NED), a hydraulic model study was conducted by the U. S. Army Engineer Waterways Experiment Station (WES) to:

- a. Study wave and current conditions in the harbor entrance both with and without the proposed improvements installed in the model.
- b. Develop remedial plans for the alleviation of undesirable wave and current conditions as found necessary.
- c. Determine if design modifications of the proposed plan could be made that would reduce construction costs significantly and still provide adequate wave protection.

PART II: THE MODEL

Design of Model

6. The Wells Harbor model (Figure 3) was constructed to an undistorted linear scale of 1:50, model to prototype. Scale selection was based on such factors as:

- a. Depth of water required in the model to prevent excessive bottom friction.
- b. Absolute size of model waves.
- c. Available shelter dimensions and area required for model construction.
- d. Efficiency of model operation.
- e. Available wave-generating and wave-measuring equipment.
- f. Model construction costs.

A geometrically undistorted model was necessary to ensure accurate reproduction of short-period wave patterns. Following selection of the linear scale, the model was designed and operated in accordance with Froude's model law.[2] The scale relations, model to prototype, used for design and operation of the model were as follows:

Characteristic	Dimension*	Scale Relation
Length	L	$L_r = 1:50$
Area	L^2	$A_r = L_r^2 = 1:2,500$
Volume	L^3	$\forall_r = L_r^3 = 1:125,000$
Time	T	$T_r = L_r^{1/2} = 1:7.07$
Velocity	L/T	$V_r = L_r^{1/2} = 1:7.07$
Discharge	L^3/T	$Q_r = L_r^{5/2} = 1:17,680$

* Dimensions are in terms of length and time.

7. The proposed improvement plans for Wells Harbor included the use of rubble-mound spur dikes and breakwaters. The existing

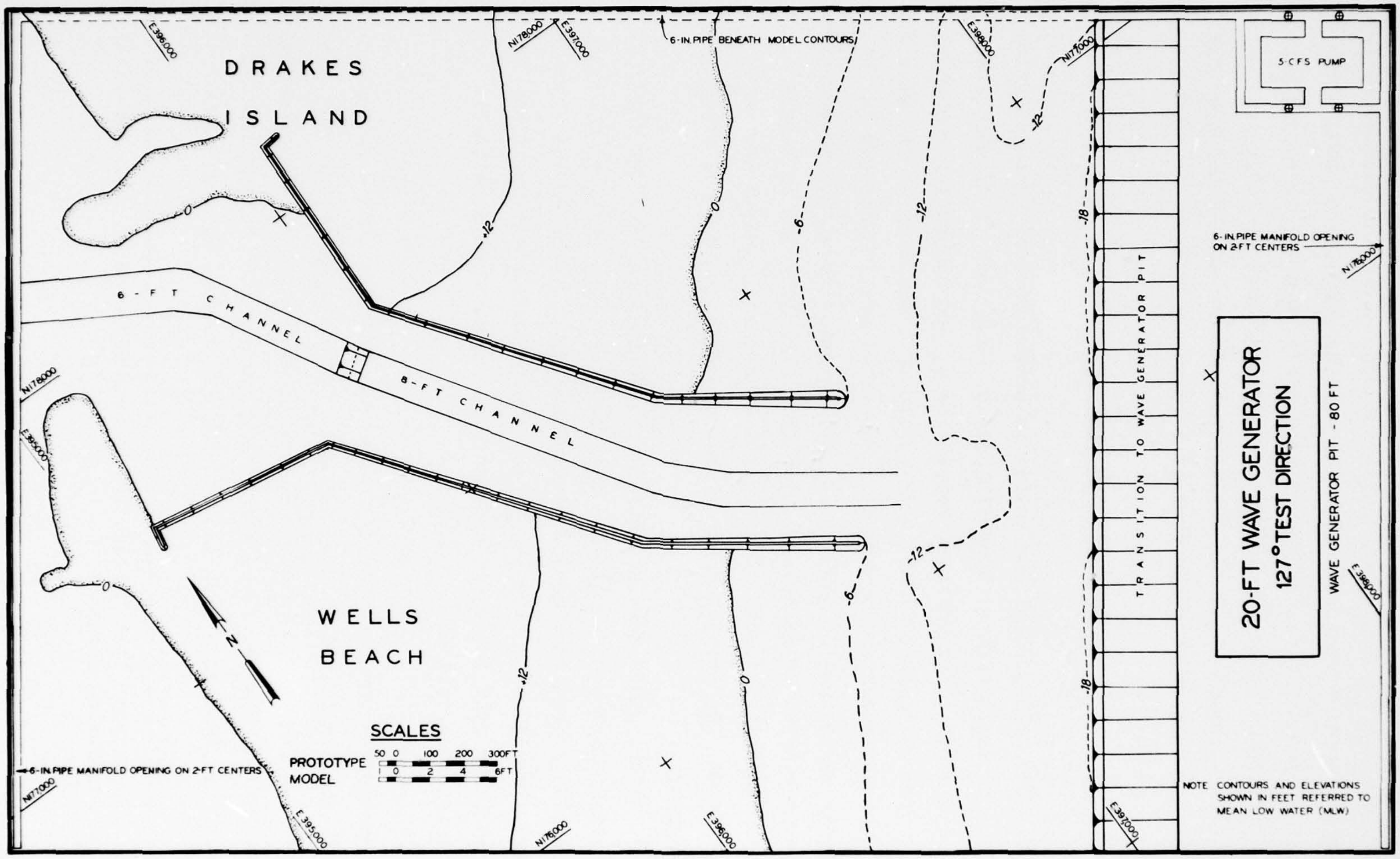

Figure 3. Model layout

breakwaters are also rubble-mound structures. Based on past experience, 1:50-scale model structures should not create sufficient scale effects to warrant geometric distortion of rock sizes in order to ensure proper transmission and reflection of wave energy. Therefore, rock size selection was based on linear scale relations and an assumed specific weight of 165 lb/ft^3 for the prototype rock.

The Model and Appurtenances

8. The model, which was molded in cement mortar, reproduced the entrance to Wells Harbor; approximately 900 and 1100 ft of shoreline south and north of the harbor, respectively; and underwater contours to an offshore depth of 18 ft with a sloping transition to the wave generator pit elevation of -80 ft. The total area reproduced in the model was approximately 4100 sq ft, representing about 0.4 square miles in the prototype. A general view of the model is shown in Figure 4. Vertical control for model construction was based on mean low water (mlw)* datum. Horizontal control was referenced to a local prototype grid system.

9. Model waves were generated by a 20-ft-long wave generator with a trapezoidal-shaped, vertical motion plunger. The vertical movement of the plunger caused a periodic displacement of water incident to this motion. The length of the stroke and the period of the vertical motion were variable over the range necessary to generate waves with the required characteristics.

10. A water-circulating system (Figure 3), consisting of 6-in. perforated-pipe water-intake and discharge manifolds, a 5-cfs pump, four valves, an orifice plate, and a differential manometer, was used in the model to reproduce steady-state flows. These flows corresponded to the maximum ebb and flood tidal flows through the harbor entrance.

11. An Automated Data Acquisition and Control System (ADACS),

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* All elevations cited herein are in feet referred to mean low water unless otherwise defined.

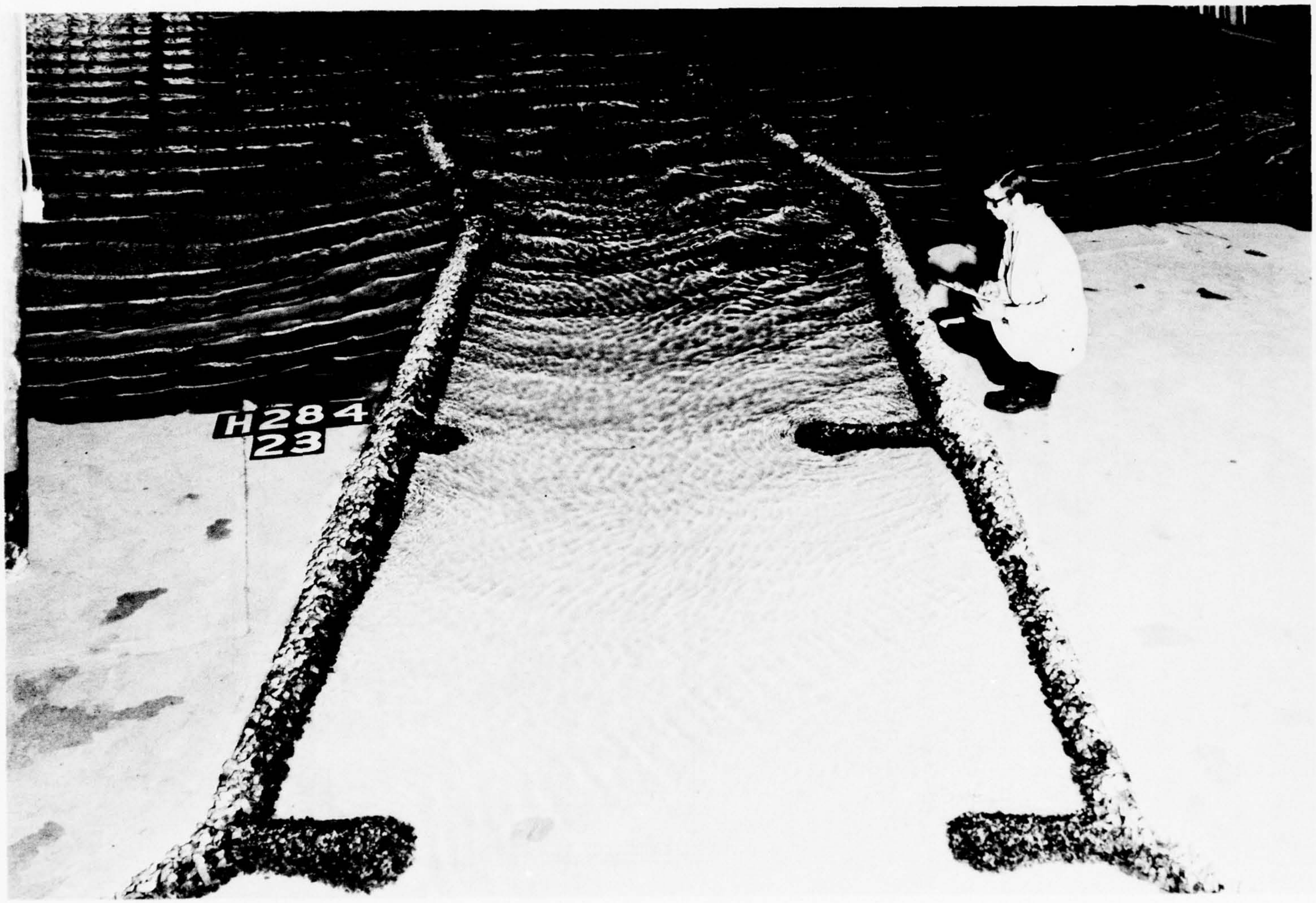

Figure 4. General view of model

designed and constructed at WES (Figure 5), was used to secure wave-height data at selected locations in the model. Basically, through the use of a minicomputer, ADACS recorded onto magnetic tape the electrical output of parallel-wire, resistance-type wave gages that measured the change in water-surface elevation with respect to time. The magnetic tape output of ADACS was then analyzed to obtain the wave-height data.

12. A 2-ft (horizontal) solid layer of fiber wave absorber was placed around the inside perimeter of the model to dampen any wave energy that might otherwise be reflected from the model walls. In addition, guide vanes were placed along the wave generator sides to ensure proper formation of the wave train incident to the model contours.

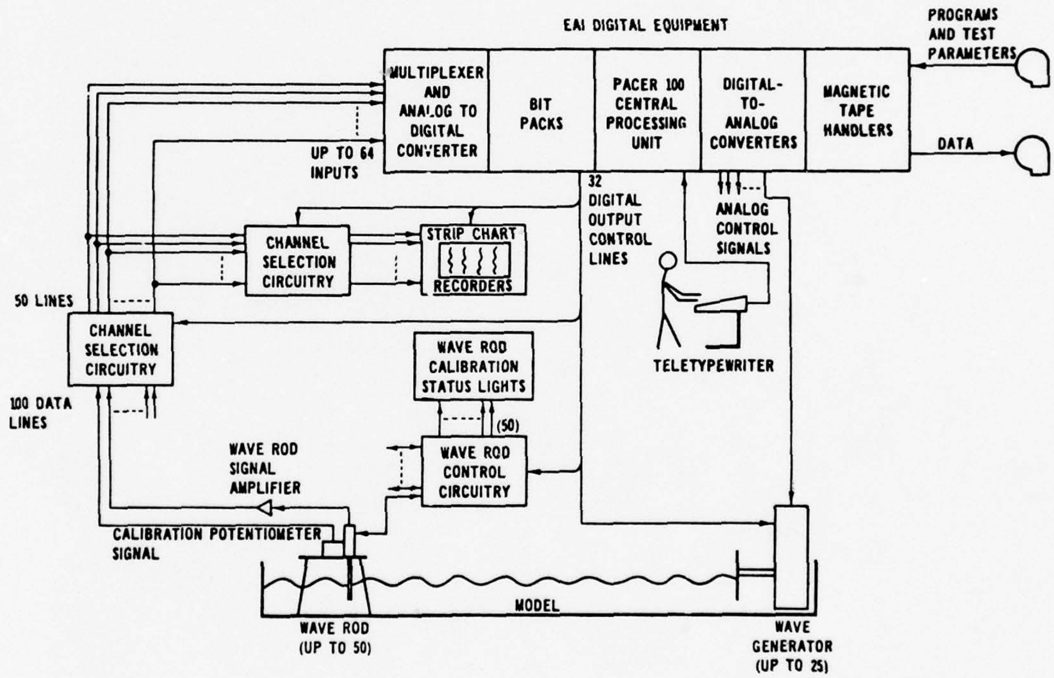

Figure 5. Automated Data Acquisition and Control System (ADACS)

PART III: TEST CONDITIONS AND PROCEDURES

Selection of Test Conditions

Still-water level

13. Still-water levels (swl's) for harbor wave-action models are selected so that various wave-induced phenomena dependent on water depths are accurately reproduced in the model. These phenomena include the refraction of waves in the harbor area, overtopping of harbor structures by waves, reflection of wave energy from harbor structures, and transmission of wave energy through porous structures.

14. It was desirable to select a model swl that closely approximated the higher water stages which normally occur in the prototype for the following reasons:

- a. The maximum amount of wave energy reaching a coastal area normally occurs during the higher water phase of the local tide cycle.
- b. Most storms moving onshore are characteristically accompanied by a higher water level due to wind tide and shoreward mass transport.
- c. The selection of a high swl helps minimize model scale effects due to viscous bottom friction.

15. Prototype data[1] indicate that maximum velocities through the harbor entrance during the ebb phase of the tidal cycle occur at a swl of +4.5 ft, and maximum velocities during the flood phase occur at a +6.8 ft swl. Therefore, swl's of +4.5 and +6.8 ft were selected for model testing of maximum ebb and flood tidal flow conditions. In addition, a swl of +8.6 ft representing slack tidal flow at mean high water (mhw) also was selected for model tests.

Factors influencing selection of test wave characteristics

16. In planning the testing program for a model investigation of harbor wave-action problems, it is necessary to select dimensions and directions for the test waves that will allow a realistic test of proposed improvement plans and an accurate evaluation of the elements

of the various proposals. Surface wind waves are generated primarily by the interactions between tangential stresses of wind flowing over water, resonance between the water surface and atmospheric turbulence, and interactions between individual wave components. The height and period of the maximum wave that can be generated by a given storm depends on the wind speed, the length of time that wind of a given speed continues to blow, and the water distance (fetch) over which the wind blows. Selection of test conditions entails evaluation of such factors as:

- a. The fetch and decay distances (the latter being the distance over which waves travel after leaving the generating area) for various directions from which waves can attack the problem area.
- b. The frequency of occurrence and duration of storm winds from the different directions.
- c. The alignment, size, and relative geographic position of the navigation entrance to the harbor.
- d. The alignments, lengths, and locations of various reflecting surfaces inside the harbor.
- e. The refraction of waves caused by differentials in depth in the area seaward of the harbor, which may create either a concentration or diffusion of wave energy at the harbor site.

Wave refraction

17. When waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except wave period. The most important transformations with respect to selection of test wave characteristics are the changes in wave height and direction of travel due to the phenomenon referred to as wave refraction. The change in wave height and direction can be determined by plotting refraction diagrams and calculating refraction coefficients. These diagrams are constructed by plotting the position of wave orthogonals (lines drawn perpendicular to wave crests) from deep water into shallow water. If it is assumed that waves do not break and there is no lateral flow of energy along the wave crest, the ratio between the wave height in deep water (H_o) and the wave height at any point in shallow water (H) is inversely proportional to the square root of the ratio of the corresponding orthogonal spacings (b_o and b), or $H/H_o = K_s(b_o/b)^{1/2}$.

The quantity $(b_o/b)^{1/2}$ is the refraction coefficient, K_r ; K_s is the shoaling coefficient. Thus, the refraction coefficient multiplied by the shoaling coefficient gives a conversion factor for transfer of deepwater wave heights to shallow-water values. The shoaling coefficient, which is a function of wavelength and water depth, can be obtained from Reference 3.

18. A wave refraction study for Wells Harbor was conducted in 1975 for the critical directions of wave approach using computer facilities at WES. These data were examined to determine the shallow-water wave height and refracted wave direction at the -18 ft contour. This analysis indicates that most deepwater waves arrive at the harbor entrance directly down the axis of the channel (127°), and that no deepwater waves arrive more than 15° to the north of the channel axis (112°) nor 20° from the south (147°). Based on this information, the wave generator position selected for use in the model was 127° which represented deepwater waves approaching from east clockwise through south-southeast.

Prototype wave data and selection of test waves

19. Measured prototype wave data on which a comprehensive statistical analysis of wave conditions could be based were unavailable for the Wells Harbor area. However, statistical deepwater wave hindcast data representative of this area were obtained from Reference 4. These data (summarized in Table 1) represent estimated durations and magnitudes of deepwater waves approaching Wells Harbor from the various directions. The data in Table 1 were converted to shallow-water values (Table 2) by the application of refraction and shoaling coefficients. The characteristics of test waves used in the model were selected from these shallow-water values and are shown in the following tabulation.

	Selected Test Waves	
Selected Shallow-Water Test Direction	Period sec	Height ft
127 deg	5	4, 7
	8	6, 10, 14
	11	6, 10, 14
	14	4, 7, 10
	17	4, 7

Tidal flows and velocities

20. Data on tidal flow through the inlet (in the vicinity of town dock) were obtained from References 1 and 5. Tidal velocities of 1.9 and 2.1 fps were selected as being representative of maximum flood and ebb conditions, respectively.

Analysis of Model Data

21. Relative merits of the various plans tested were evaluated by:

   a. Comparison of wave heights at selected locations in the harbor entrance.

   b. Comparison of current patterns and magnitudes in the harbor entrance.

   c. Visual observations and wave pattern photographs.

In analyzing the wave-height data, the average height of the highest one third of the waves recorded at each gage location was selected. All wave heights thus selected then were adjusted to compensate for wave-height attenuation due to viscous bottom friction in the model by application of Keulegan's equation.[6] From this equation, reduction of wave heights in the model can be calculated as a function of water depth, width of wave front, wave period, water viscosity, and distance of wave travel.

PART IV: TESTS AND RESULTS

The Tests

Existing conditions

22. Prior to tests of various improvement plans, comprehensive tests were conducted for existing conditions. Wave-height data were obatined at various locations in the jettied entrance (Plate 1) for the test waves listed in paragraph 19. Wave-induced current patterns and magnitudes, in conjunction with maximum ebb and flood tidal flows, and wave pattern photographs also were secured for representative test waves.

Improvement plans

23. Wave heights, current patterns and magnitudes, and wave pattern photographs were secured for three improvement plans. Two of the improvement plans consisted of installation of stone spur dikes in the jettied entrances, and the other plan included a breakwater attached to the north jetty in conjunction with spur dikes. Brief descriptions of the test plans are presented in the following subparagraphs; details are presented in Plates 2-4. Typical breakwater and spur dike sections are shown in Plate 5.

a. Plan 1 (Plate. 2) consisted of the originally proposed spur dike design. This plan included a total of 10 spur dikes in the jettied entrance that reduced the controlling width between jetties from 400 ft to 280 ft. The crown elevation of the spur dikes was +8.6 ft and the side slopes were 1V:1.5H on the trunk sections and 1V:2H on the head sections.

b. Plan 2 (Plate 3) entailed the elements of plan 1 with the following modifications

Spur-Dike Number When Approaching from Sea	Spur-Dike Location North/South Jetty	Change to Spur Dike
First	South	Removed
Second	South	1V:3H slope installed on seaward side

(Continued)

Spur-Dike Number When Approaching from Sea	Spur-Dike Location North/South Jetty	Change to Spur Dike
First	North	Crown elevation raised to +16 ft; 1V:3H slope installed on seaward side
Second	North	1V:3H slope installed on seaward side
Third	North	Removed

c. Plan 3 (Plate 4) involved the elements of plan 1 except the first spur dike attached to the south jetty and the first and third spur dikes attached to the north jetty (approaching from the sea) were removed. A 565-ft-long breakwater was attached to the north jetty and oriented 45° to the south of the north jetty alignment.

Wave-height tests

24. Wave-height tests for plans 1 and 3 were conducted using all the test waves listed in paragraph 19 for maximum ebb and flood tidal flows and for slack water. As an expedient, wave-height tests were conducted for plan 2 only for representative test waves. Wave gage locations for the various test plans are shown in Plates 2-4.

Current pattern and magnitude tests

25. Wave-induced current patterns and magnitudes in conjunction with maximum ebb and maximum flood tidal flows were determined at selected locations in the jettied entrance by timing the progress of a dye tracer relative to a known distance on the model. These tests were conducted for representative test waves with plans 1 and 3 installed.

Movie

26. A 20-min movie of the Wells Harbor model showing tests of existing conditions and plans 1 and 3 was secured and forwarded to NED for use in public meetings. Included in the movie footage were the following:

a. A general view of Wells Harbor model.

b. Wave conditions in the entrance for 8-sec, 10-ft waves with maximum ebb flow conditions.

c. Current patterns in the entrance for maximum ebb flow conditions.

d. Wave conditions in the entrance for 8-sec, 6-ft waves with maximum flood-flow conditions.

e. Current patterns in the entrance for maximum flood-flow conditions.

f. Wave conditions in the entrance for 8-sec, 10-ft waves with slack water.

Test Results

27. In evaluating test results, the relative merits of each plan were based on an analysis of measured wave heights and current patterns and magnitudes. Model wave heights (significant wave height or $H_{1/3}$) were tabulated to show measured values at selected locations. Current patterns and magnitudes were superimposed on wave pattern photographs for the corresponding plan and wave condition tested.

Existing conditions

28. Wave-height measurements obtained for existing conditions are presented in Table 3. Maximum wave heights obtained in the jettied entrance were 8.7, 9.2, and 10.6 ft at the entrance (gage 1); 5.2, 6.1, and 7.7 ft at the bend in the jetties (gage 3); and 1.3, 2.1, and 2.7 ft where the jetties start widening (gage 6) for the +4.5, +6.8, and 8.6 ft swl's, respectively.

29. Current patterns and magnitudes for existing conditions are shown in Photos 1-10 for maximum ebb (+4.5 ft swl) and maximum flood (+6.8 ft swl) tidal flow conditions. Maximum velocities for ebb flow conditions in the jettied entrance ranged from 2.4 fps for 14-sec, 7-ft and 17-sec, 4-ft test waves to 3.5 fps for 11-sec, 10-ft test waves. For flood-flow conditions, maximum velocities between the jetties ranged from 2.4 fps for 8-sec, 10-ft and 17-sec, 4-ft test waves to 3.5 fps for 11-sec, 10-ft test waves. Wave-induced current patterns and magnitudes secured for slack water (+8.6 ft swl) indicated no definite current patterns and velocities obtained were very small. Typical wave patterns obtained for existing conditions are shown in Photos 1-15.

30. Using wave heights obtained for existing conditions in the vicinity of the proposed spur dikes, stone sizes were calculated by WES

personnel using design procedures from Reference 3. Based on a 10.6-ft design wave height, the first three spur dikes on each side of the channel required the following stone sizes for stability: armor layer, 4.2 tons, first underlayer, 842 lb; core stone, 42 lb. The design of the remaining spur dikes was based on a 5-ft design wave, and the following stone sizes were required: armor layer, 884 lb; first underlayer, 88 lb, core stone, 4.5 lb.

Improvement plans

31. Results of wave-height tests with plan 1 installed in the model are presented in Table 4. Maximum wave heights obtained in the jettied entrance were 9.2, 10.4, and 10.7 ft at the entrance (gage 1); 4.7, 6.6, and 7.3 ft at the bend in the jetties (gage 3); and 1.0, 1.2, and 1.9 ft where the jetties start widening (gage 6) for the +4.5, +6.8, and +8.6 ft swl's, respectively.

32. Current patterns and magnitudes for plan 1 are shown in Photos 16-25 for maximum ebb and maximum flood tidal flow conditions. Maximum velocities in the jettied entrance ranged from 3.2 fps to 3.4 fps for test waves for maximum ebb flow conditions. For flood conditions, maximum velocities between the jetties ranged from 2.8 fps for 5-sec, 7-ft test waves to 4.7 fps for 11-sec, 10-ft test waves. Typical wave patterns obtained for plan 1 are shown in Photos 16-30.

33. Wave-height measurements obtained with plan 2 installed are presented in Table 5. Maximum wave heights obtained in the jettied entrance were 8.8, 10.2, and 11.1 ft at the entrance (gage 1) and 4.4, 5.7, and 6.6 ft at the bend in the jetties (gage 3) for the +4.5, +6.8, and +8.6 ft swl's, respectively.

34. Typical wave patterns for plan 2 are shown in Photos 31 and 32 for slack water (+8.6 ft swl).

35. Results of wave-height tests with plan 3 installed are presented in Table 6. Maximum wave heights obtained in the jettied entrance were 6.7, 9.4, and 9.2 ft at the entrance (gage 1A); 1.4, 1.9, and 1.8 ft at the bend in the jetties (gage 3); and 0.6, 0.3, and 0.6 ft where the jetties start widening (gage 6) for the +4.5, +6.8, and +8.6 ft swl's, respectively.

36. Current patterns and magnitudes for plan 3 are shown in Photos 33-38 for maximum ebb and flood tidal flow conditions and representative test waves. Maximum velocities in the jettied entrance for ebb flow conditions ranged from 3.1 to 3.5 fps. For flood-flow conditions, maximum velocities between the jetties ranged from 3.4 to 3.6 fps. Typical wave patterns for plan 3 are shown in Photos 33-41.

Discussion of test results

37. Test results obtained for existing conditions indicate breaking waves in the harbor entrance for certain incident wave conditions. This problem is compounded by the presence of ebb tidal currents which tend to make the incident waves steepen and change direction. In general, navigation conditions are poor when waves are moderate to large.

38. A comparison of wave heights obtained for existing conditions and plan 1 (Table 7) revealed that the plan 1 configuration, in general, slightly increased wave heights at gage 1, had little effect on wave heights at gages 2 and 3, and reduced wave heights at gages 4-8. It appears that the net overall effect of plan 1 is an improvement of entrance wave conditions.

39. A comparison of current magnitudes for existing conditions and plan 1 indicates that the decreased channel width of plan 1 increased ebb current magnitudes an average of 51 percent and flood current magnitudes an average of 57 percent. It should be noted however, that the model has a fixed bed (i.e., the bottom is not allowed to scour) and that in the prototype, the channel bottom will scour, increasing the cross-sectional area, thereby offsetting this initial increase in current velocities. Whether velocities (following installation of the spur dikes) will be comparable to those presently existing, and whether the channel will scour sufficiently to maintain navigable depths is beyond the scope of this model investigation. However, qualitative indications are that the spur dikes will be beneficial in reducing maintenance dredging. Current patterns observed in the model indicate that some shoaling is likely to occur between the spur dikes, especially the larger ones attached to the south jetty.

40. Visual observations of current patterns for plan 1 revealed

that the first spur dike attached to the south jetty and the third spur dike attached to the north jetty (approaching from the sea) had little effect on current patterns and could be removed without compromising effectiveness.

41. A comparison of wave heights obtained for existing conditions, plan 1, and plan 2 (Table 8) indicates that plan 2 wave heights were generally similar to those for plan 1 and were slightly greater than those for existing conditions at gage 1. Wave heights were about the same for existing conditions and both plans at gages 2 and 3. Raising the crown elevation and/or flattening the seaward slope of the spur dikes apparently had little effect on wave heights.

42. Test results for plan 3 indicated that wave heights in the jettied entrance were significantly decreased due to the installation of the breakwater and navigation conditions would be substantially improved. Current patterns and magnitudes obtained for plan 3 were similar to those obtained for plan 1.

PART V: CONCLUSIONS

43. Based on the results of the hydraulic model investigation reported herein, it is concluded that:

- a. During periods of moderate to large wave attack, the existing harbor entrance experiences hazardous navigation conditions due to breaking waves and interaction of waves with tidal currents.
- b. The originally proposed spur-dike configuration (plan 1) will slightly increase wave heights in the outer entrance (gage 1), will have little effect on wave heights between the outer entrance and the bend in the jetties, and will reduce wave heights through the remainder of the jettied entrance.
- c. Changing the spur-dike cross sections at the entrance (plan 2) will not reduce wave heights at that location.
- d. Removal of the first spur dike attached to the south jetty and the third spur dike attached to the north jetty (approaching from the sea) will not compromise design effectiveness.
- e. Shoaling may occur between the spur dikes, particularly the longer ones attached to the south jetty.
- f. The installation of a breakwater (plan 3) will substantially improve wave conditions in the jettied entrance over those for existing conditions or any of the spur-dike plans tested.
- g. Whether tidal velocities following installation of the proposed spur dikes will be sufficient to maintain a self-scouring channel at desired navigable depths is beyond the scope of this model investigation. However, qualitative indications are that the spur dikes will be beneficial in reducing maintenance dredging requirements.

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Table 1

Estimated Duration and Magnitude of Deepwater Waves Approaching

Wells Harbor Entrance from Various Directions

Wave Height* ft	Duration, hr/yr, per Wave Period, sec* 4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	Total
					East					
0.5- 2.0	27	199	396	157	3	--	--	--	--	782
2.0- 4.0	49	37	137	224	55	8	--	--	--	510
4.0- 6.0	28	35	44	77	80	24	1	--	--	289
6.0- 8.0	3	40	16	35	41	33	3	--	--	171
8.0-10.0	--	28	24	17	35	15	1	--	--	120
10.0-12.0	--	15	21	12	9	12	7	--	--	76
12.0-14.0	--	--	27	7	3	4	1	--	--	42
14.0-16.0	--	--	11	3	--	4	--	1	--	19
16.0-18.0	--	--	1	9	8	1	1	--	--	20
18.0-20.0	--	--	1	1	--	3	--	--	--	5
20.0-25.0	--	--	--	4	--	--	3	--	--	7
Total	107	354	678	546	234	104	17	1		2041
					East-Southeast					
0.5- 2.0	33	115	175	21	--	--	--	--	--	344
2.0- 4.0	27	35	65	111	20	1	--	--	--	259
4.0- 6.0	16	24	11	23	20	9	9	--	--	112
6.0- 8.0	1	29	12	9	11	17	1	--	--	80
8.0-10.0	--	15	8	3	4	8	--	--	--	38

(Continued)

* Wave height and wave period groupings include the lower but not the upper values.

Table 1 (Continued)

Wave Height* ft	Duration, hr/yr, per Wave Period, sec* 4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	Total
				East-Southeast (Continued)						
10.0-12.0	--	5	9	1	1	1	1	--	--	18
12.0-14.0	--	--	8	4	--	--	--	--	--	12
14.0-16.0	--	--	5	--	--	1	--	--	--	6
16.0-18.0	--	1	4	1	--	--	1	--	--	7
18.0-20.0	--	--	1	5	--	--	--	--	--	6
20.0-25.0	--	--	--	5	1	--	3	--	--	9
Total	77	224	298	183	57	37	15			891
				Southeast						
0.5- 2.0	23	61	51	4	--	--	--	--	--	139
2.0- 4.0	24	33	35	25	24	1	--	--	--	118
4.0- 6.0	21	32	15	15	16	3	1	--	--	103
6.0- 8.0	--	23	5	1	13	12	--	--	--	54
8.0-10.0	--	12	1	3	5	3	--	--	--	48
10.0-12.0	--	8	7	--	1	1	5	1	--	23
12.0-14.0	--	--	7	1	--	--	3	--	--	11
14.0-16.0	--	--	4	3	1	--	1	--	1	10
16.0-18.0	--	--	1	3	--	--	--	--	--	4
18.0-20.0	--	--	--	--	--	--	--	--	1	1
20.0-25.0	--	--	--	3	--	3	--	--	--	6
25.0-30.0	--	--	--	1	--	--	--	--	--	1
Total	68	169	126	59	60	23	10	1	2	518

(Continued)

Table 1 (Concluded)

Wave Height* ft	Duration, hr/yr, per Wave Period, sec*									Total
	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	
					South-Southeast					
0.5- 2.0	12	64	21	12	--	--	--	--	--	109
2.0- 4.0	28	13	9	23	23	--	--	--	--	96
4.0- 6.0	21	12	12	7	13	17	--	--	--	82
6.0- 8.0	1	23	7	5	3	8	7	--	--	54
8.0-10.0	--	8	3	--	1	7	4	--	--	23
10.0-12.0	--	1	3	--	3	7	--	--	--	14
12.0-14.0	--	--	4	--	--	--	1	--	--	5
14.0-16.0	--	--	4	--	--	--	--	--	--	4
Total	62	121	63	47	43	39	12			387

Table 2

Estimated Duration and Magnitude of Shallow-Water Waves Approaching Wells Harbor Entrance from Various Directions

Wave Height* ft	Duration, hr/yr, per Wave Period, sec*									
	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	Total
					East					
0.5- 2.0	27	199	396	381	58	8	--	--	--	1069
2.0- 4.0	49	72	137	112	80	57	4	--	--	511
4.0- 6.0	28	40	60	36	76	27	8	--	--	275
6.0- 8.0	3	28	24	--	12	8	1	1	--	77
8.0-10.0	--	15	21	13	8	4	1	--	--	62
10.0-12.0	--	--	38	4	--	--	--	--	--	42
12.0-14.0	--	--	1	--	--	--	3	--	--	4
14.0-16.0	--	--	1	--	--	--	--	--	--	1
Total	107	354	678	546	234	104	17	1		2041
					East-Southeast					
0.5- 2.0	33	115	175	132	--	1	--	--	--	456
2.0- 4.0	27	35	65	23	40	26	10	--	--	226
4.0- 6.0	16	24	23	12	15	9	1	--	--	100
6.0- 8.0	1	29	8	5	1	1	--	--	--	45
8.0-10.0	--	15	17	1	--	--	1	--	--	34
10.0-12.0	--	5	5	5	--	--	--	--	--	15
12.0-14.0	--	--	5	5	1	--	3	--	--	14
14.0-16.0	--	1	--	--	--	--	--	--	--	1
Total	77	224	298	183	57	37	15			891

(Continued)

* Wave height and wave period groupings include the lower but not the upper values.

(Sheet 1 of 2)

Table 2 (Concluded)

Wave Height* ft	Duration, hr/yr, per Wave Period, sec* 4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	Total
					Southeast					
0.5- 2.0	23	61	51	29	24	1	--	--	--	189
2.0- 4.0	24	33	50	16	16	15	1	--	--	155
4.0- 6.0	21	32	5	3	18	4	8	1	--	92
6.0- 8.0	--	23	8	4	1	--	1	--	1	38
8.0-10.0	--	12	7	3	1	--	--	--	1	24
10.0-12.0	--	8	5	--	--	3	--	--	--	16
12.0-14.0	--	--	--	3	--	--	--	--	--	3
14.0-16.0	--	--	--	1	--	--	--	--	--	1
Total	68	169	126	59	60	23	10	1	2	518
					South-Southeast					
0.5- 2.0	12	64	21	35	23	17	--	--	--	172
2.0- 4.0	28	13	21	12	17	22	11	--	--	124
4.0- 6.0	21	12	7	--	3	--	1	--	--	44
6.0- 8.0	1	23	6	--	--	--	--	--	--	30
8.0-10.0	--	9	4	--	--	--	--	--	--	13
10.0-12.0	--	--	4	--	--	--	--	--	--	4
Total	62	121	63	47	43	39	12			387

(Sheet 2 of 2)

Table 3

Wave Heights for Existing Conditions

Direction 127°

Test Wave		Wave Height, ft							
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8
				swl = +4.5 ft (Maximum Ebb)					
5.0	4.0	8.7	5.6	4.2	2.2	0.8	0.4	0.3	<0.1
	7.0	7.1	7.6	5.0	4.1	1.3	0.8	0.7	<0.1
8.0	6.0	7.9	7.9	5.0	3.5	1.8	1.0	0.7	0.2
	10.0	8.0	8.3	5.2	3.6	2.0	1.3	0.9	0.4
	14.0	6.9	6.5	4.6	3.0	1.9	1.2	0.9	0.3
11.0	6.0	7.3	6.4	4.9	3.2	2.1	1.2	0.5	0.3
	10.0	6.9	6.0	3.8	3.8	2.2	1.3	0.6	0.3
	14.0	6.9	6.4	3.8	3.3	1.8	1.1	0.4	0.2
14.0	4.0	8.0	6.4	4.8	3.4	1.7	1.0	0.5	0.3
	7.0	7.6	7.1	5.1	3.8	1.8	1.3	0.8	0.3
	10.0	7.4	6.1	4.3	3.6	1.8	0.8	0.6	0.2
17.0	4.0	7.2	5.8	4.3	3.8	2.1	1.2	0.9	0.4
	7.0	6.5	6.1	4.9	3.3	1.7	1.1	0.5	0.3
				swl = +6.8 ft (Maximum Flood)					
5.0	4.0	5.3	5.9	4.2	4.1	2.5	1.2	0.5	0.4
	7.0	9.0	7.0	4.6	1.2	1.0	0.4	0.4	0.2
8.0	6.0	9.1	9.4	5.8	2.1	2.6	1.0	0.8	0.4
	10.0	7.2	5.4	6.1	3.5	3.0	1.8	0.9	0.6
	14.0	7.2	5.3	4.3	2.4	2.6	1.5	1.3	0.6
11.0	6.0	8.8	4.6	3.4	2.5	1.5	1.0	0.7	0.3
	10.0	7.1	4.2	2.7	2.7	1.9	0.7	0.7	0.5
	14.0	7.2	4.8	3.3	2.3	1.8	0.8	0.7	0.5
14.0	4.0	9.2	8.3	5.6	4.9	2.6	2.1	1.3	0.7
	7.0	6.9	5.5	3.6	1.6	1.4	1.0	0.6	0.3
	10.0	7.3	5.5	3.7	3.7	1.6	1.5	0.9	0.6
17.0	4.0	8.1	5.3	4.4	3.0	2.5	1.3	1.1	0.5
	7.0	7.0	4.1	3.5	2.5	1.4	0.9	0.7	0.3

(Continued)

Table 3 (Concluded)

Test Wave		Wave Height, ft							
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8
		swl = +8.6 ft (Slack Water)							
5.0	4.0	4.5	6.7	4.2	4.0	1.8	0.5	1.0	0.4
	7.0	7.5	10.1	4.1	1.7	0.8	0.5	0.4	0.1
8.0	6.0	8.4	10.0	6.6	4.4	3.5	2.2	1.6	0.5
	10.0	10.6	10.3	7.7	4.8	4.0	2.7	2.0	1.0
	14.0	8.1	7.2	6.7	3.6	2.9	2.2	1.9	0.9
11.0	6.0	9.9	7.3	3.3	2.7	2.1	1.0	1.3	0.5
	10.0	8.3	5.3	5.3	3.4	1.6	1.0	0.8	0.6
	14.0	8.3	6.3	4.3	3.9	2.2	1.3	1.1	0.6
14.0	4.0	8.7	7.8	4.5	4.6	2.4	1.8	1.1	0.8
	7.0	7.7	8.2	5.5	3.7	2.6	1.7	1.1	0.7
	10.0	8.3	6.5	4.1	4.0	2.7	1.8	1.1	0.7
17.0	4.0	10.2	8.1	6.5	4.3	3.3	2.3	2.0	0.9
	7.0	7.6	5.2	4.7	3.9	2.4	1.3	1.1	0.5

Table 4

Wave Heights for Plan 1

Direction 127°

Test Wave Period sec	Test Wave Height ft	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8
swl = +4.5 ft (Maximum Ebb)									
5.0	4.0	8.9	4.2	3.1	1.7	0.9	0.5	0.3	0.2
	7.0	9.2	5.4	3.1	1.5	1.3	0.9	0.5	0.2
8.0	6.0	8.3	7.4	3.8	2.8	1.8	1.0	0.6	0.2
	10.0	8.2	7.3	4.1	2.9	1.8	1.0	0.7	0.2
	14.0	7.6	7.3	3.4	2.7	1.4	0.7	0.5	0.2
11.0	6.0	6.7	6.1	3.3	2.1	1.6	0.9	0.5	0.3
	10.0	6.4	6.4	3.7	2.4	1.5	0.8	0.3	0.2
	14.0	6.9	5.7	3.3	2.2	1.6	0.8	0.4	0.2
14.0	4.0	6.5	4.9	4.7	2.3	1.4	0.9	0.5	0.2
	7.0	6.7	5.8	3.8	2.5	1.3	0.9	0.6	0.3
	10.0	6.4	5.6	3.5	2.1	1.1	0.7	0.5	0.2
17.0	4.0	6.4	5.9	3.0	2.0	1.4	0.9	0.4	0.2
	7.0	5.9	6.0	3.8	2.7	1.6	0.8	0.5	0.3
swl = +6.8 ft (Maximum Flood)									
5.0	4.0	5.0	4.9	2.9	1.4	0.6	0.2	0.1	<0.1
	7.0	7.0	7.6	5.7	4.1	1.7	0.7	0.7	0.3
8.0	6.0	10.4	8.4	6.6	2.8	1.4	1.2	0.7	0.3
	10.0	9.2	6.4	4.3	2.5	0.6	0.8	0.5	0.2
	14.0	8.7	6.2	4.0	2.1	0.4	0.5	0.4	0.2
11.0	6.0	8.0	3.5	3.5	1.5	0.9	0.7	0.4	0.1
	10.0	7.5	4.7	3.0	1.4	0.8	0.6	0.3	0.1
	14.0	7.1	5.3	3.1	1.4	0.8	0.4	0.3	0.1
14.0	4.0	8.5	6.7	3.3	2.7	1.4	0.9	0.6	0.3
	7.0	7.2	4.2	3.1	1.6	1.1	0.5	0.3	0.2
	10.0	7.1	4.5	2.6	1.6	1.1	0.5	0.4	0.2
17.0	4.0	8.3	5.7	2.9	2.1	0.8	0.5	0.3	0.2
	7.0	7.1	4.2	1.8	1.5	0.6	0.4	0.1	<0.1

(Continued)

Table 4 (Concluded)

Test Wave Period sec	Test Wave Height ft	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8
				swl = +8.6 ft (Slack Water)					
5.0	4.0	4.7	6.6	4.1	2.9	1.6	0.4	0.6	0.1
	7.0	8.2	9.5	5.6	3.0	1.4	0.4	0.6	0.2
8.0	6.0	9.8	10.4	7.3	4.4	4.0	1.8	1.0	0.4
	10.0	10.2	9.9	6.7	3.4	3.4	1.5	0.8	0.3
	14.0	9.2	8.0	5.2	2.3	2.1	0.8	0.4	0.2
11.0	6.0	10.7	7.7	5.7	2.9	2.1	1.2	1.1	0.5
	10.0	9.0	5.9	4.3	2.6	1.5	0.9	0.6	0.3
	14.0	8.8	6.0	4.1	2.6	1.9	1.0	0.8	0.3
14.0	4.0	8.9	7.0	4.9	3.1	1.9	1.9	1.0	0.6
	7.0	7.9	7.4	5.5	2.8	2.1	1.5	0.9	0.5
	10.0	8.0	7.0	4.8	3.2	1.9	1.2	0.8	0.5
17.0	4.0	10.0	8.0	5.8	3.6	3.0	1.6	1.0	0.4
	7.0	7.9	6.0	4.5	4.2	2.1	1.0	0.8	0.4

(Sheet 2 of 2)

Table 5

Wave Heights for Plan 2

Direction 127°

Test Wave		Wave Height, ft			
Period sec	Height ft	Gage 1	Gage 2	Gage 3	Gage 4
swl = +4.5 ft (Maximum Ebb)					
5.0	7.0	7.7	5.7	3.7	1.8
8.0	6.0	8.8	8.1	4.4	2.8
	14.0	7.4	4.7	3.9	2.8
11.0	10.0	6.6	6.3	3.7	2.2
swl = +6.8 ft (Maximum Flood)					
5.0	7.0	7.7	7.4	5.5	3.0
8.0	6.0	10.2	8.5	5.7	2.9
	10.0	8.4	6.0	4.3	2.5
	14.0	8.5	5.8	3.3	2.0
11.0	10.0	8.0	4.4	2.6	1.2
swl = +8.6 ft (Slack Water)					
5.0	7.0	8.5	8.2	4.2	3.0
8.0	6.0	10.4	9.8	6.6	4.3
	14.0	9.0	7.1	5.4	2.9
11.0	6.0	11.1	8.0	5.9	3.4
	10.0	8.8	5.6	3.7	2.2

Table 6

Wave Heights for Plan 3

Direction 127°

Test Wave		Wave Height, ft							
Period sec	Height ft	Gage 1A	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7
		swl = +4.5 ft (Maximum Ebb)							
5.0	4.0	1.8	0.3	0.9	<0.1	0.3	0.1	<0.1	<0.1
	7.0	3.7	0.7	1.8	0.4	0.5	0.2	0.1	<0.1
8.0	6.0	3.8	1.3	1.5	0.6	0.7	0.5	0.3	0.1
	10.0	6.2	1.4	2.3	0.7	0.8	0.5	0.3	0.1
	14.0	5.4	1.4	2.4	0.9	0.8	0.5	0.3	0.2
11.0	6.0	5.4	1.7	2.5	1.1	0.7	0.6	0.5	0.2
	10.0	6.6	2.1	3.3	1.4	1.3	0.8	0.6	0.3
	14.0	6.7	2.0	3.3	1.2	1.4	1.0	0.5	0.3
14.0	4.0	2.9	1.2	1.3	0.5	0.4	0.2	0.1	<0.1
	7.0	4.4	1.8	1.9	0.8	0.8	0.5	0.3	0.1
	10.0	5.1	1.8	2.3	0.9	0.9	0.4	0.4	0.2
17.0	4.0	2.4	1.3	1.0	0.4	0.3	0.3	0.1	<0.1
	7.0	4.4	2.0	2.1	0.7	0.7	0.4	0.3	0.2
		swl = +6.8 ft (Maximum Flood)							
5.0	4.0	1.9	0.6	0.3	0.2	0.1	<0.1	<0.1	<0.1
	7.0	4.7	1.4	0.8	0.2	0.2	<0.1	<0.1	<0.1
8.0	6.0	3.4	1.6	0.5	0.3	0.4	0.1	0.1	0.1
	10.0	7.2	2.2	1.6	0.8	0.7	0.2	0.2	0.2
	14.0	6.2	3.4	1.1	0.5	0.6	0.1	0.1	0.1
11.0	6.0	4.4	1.9	1.0	0.8	0.4	0.3	0.2	0.1
	10.0	8.2	2.6	1.9	1.4	0.6	0.3	0.2	0.2
	14.0	9.4	2.9	2.2	1.9	0.7	0.5	0.3	0.2
14.0	4.0	2.8	1.6	0.6	0.5	0.5	0.3	0.2	0.2
	7.0	5.0	2.1	1.1	0.7	0.6	0.4	0.2	0.2
	10.0	7.7	2.5	1.8	0.8	0.7	0.4	0.2	0.2
17.0	4.0	2.5	1.5	0.7	0.3	0.5	0.1	0.1	<0.1
	7.0	4.4	2.1	0.9	0.5	0.5	0.1	0.1	0.1

(Continued)

Table 6 (Concluded)

Test Wave Period sec	Test Wave Height ft	Wave Height, ft Gage 1A	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7
		swl = +8.6 ft (Slack Water)							
5.0	4.0	2.5	0.8	0.7	0.5	0.3	0.1	<0.1	<0.1
	7.0	4.6	1.4	1.2	0.7	0.4	0.1	0.1	0.1
8.0	6.0	4.1	1.5	1.3	0.8	0.7	0.5	0.2	<0.1
	10.0	7.6	2.4	1.7	1.0	1.0	0.7	0.3	0.1
	14.0	7.2	2.2	2.6	1.2	1.1	0.7	0.3	0.1
11.0	6.0	4.5	1.5	1.7	1.4	0.9	0.5	0.3	0.3
	10.0	7.7	2.1	2.4	1.7	0.8	0.6	0.4	0.3
	14.0	9.2	2.8	2.9	1.8	0.8	0.7	0.5	0.4
14.0	4.0	2.7	2.1	1.5	0.9	0.8	0.6	0.4	0.2
	7.0	5.4	2.2	2.4	1.3	1.1	0.8	0.6	0.4
	10.0	6.3	2.7	2.5	1.0	1.0	0.7	0.6	0.4
17.0	4.0	2.4	1.3	1.4	0.6	0.8	0.5	0.2	0.1
	7.0	4.5	1.8	2.1	1.0	0.8	0.5	0.3	0.2

(Sheet 2 of 2)

Table 7

Comparison of Wave Heights for Existing Conditions and Plan 1 at Gage Locations 1-3 (13 Test Waves)

Gage No.	Times Wave Heights Increased/Decreased for Plan 1: Increased	Decreased	Maximum Wave Height: Existing Conditions	Plan 1	Percent Change	Average Wave Height: Existing Conditions	Plan 1	Percent Change
			swl = +4.5 ft (Maximum Ebb)					
1	5	7	8.7	9.2	+6	7.42	7.24	-2
2	3	10	8.3	7.4	-11	6.63	6.00	-11
3	0	13	5.2	4.7	-10	4.61	3.58	-22
			swl = +6.8 ft (Maximum Flood)					
1	7	6	9.2	10.4	+13	7.65	7.78	+2
2	7	6	9.4	8.4	-11	5.79	5.56	-4
3	4	9	6.1	6.6	+8	4.25	3.60	-15
			swl = +8.6 ft (Slack Water)					
1	10	3	10.6	10.7	+1	8.32	8.72	+5
2	6	7	10.3	10.4	+1	7.62	7.65	0
3	5	7	7.7	7.3	-5	5.19	5.27	+2

Table 8

Comparison* of Wave Heights for Existing Conditions

Plan 1 and Plan 2 at Gage Locations 1-3

Gage No.	Maximum Wave Height: Existing Conditions	Plan 1	Plan 2	Average Wave Height: Existing Conditions	Plan 1	Plan 2
			swl = +4.5 ft (Maximum Ebb)			
1	7.9	9.2	8.8	7.2	7.9	7.6
2	7.9	7.4	8.1	7.0	6.6	6.2
3	5.0	3.8	4.4	4.6	3.5	3.9
			swl = +6.8 ft (Maximum Flood)			
1	9.1	10.4	10.2	7.9	8.6	8.6
2	9.4	8.4	8.5	6.3	6.7	6.4
3	5.8	6.6	5.7	4.7	4.7	5.1
			swl = +8.6 ft (Slack Water)			
1	9.9	10.7	11.1	8.4	9.4	9.6
2	10.1	10.4	9.8	8.0	8.3	7.7
3	6.7	7.3	6.6	5.2	5.6	5.2

* Compared for plan 2 test waves only.

Photo 1. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 5-sec, 7-ft waves for maximum ebb

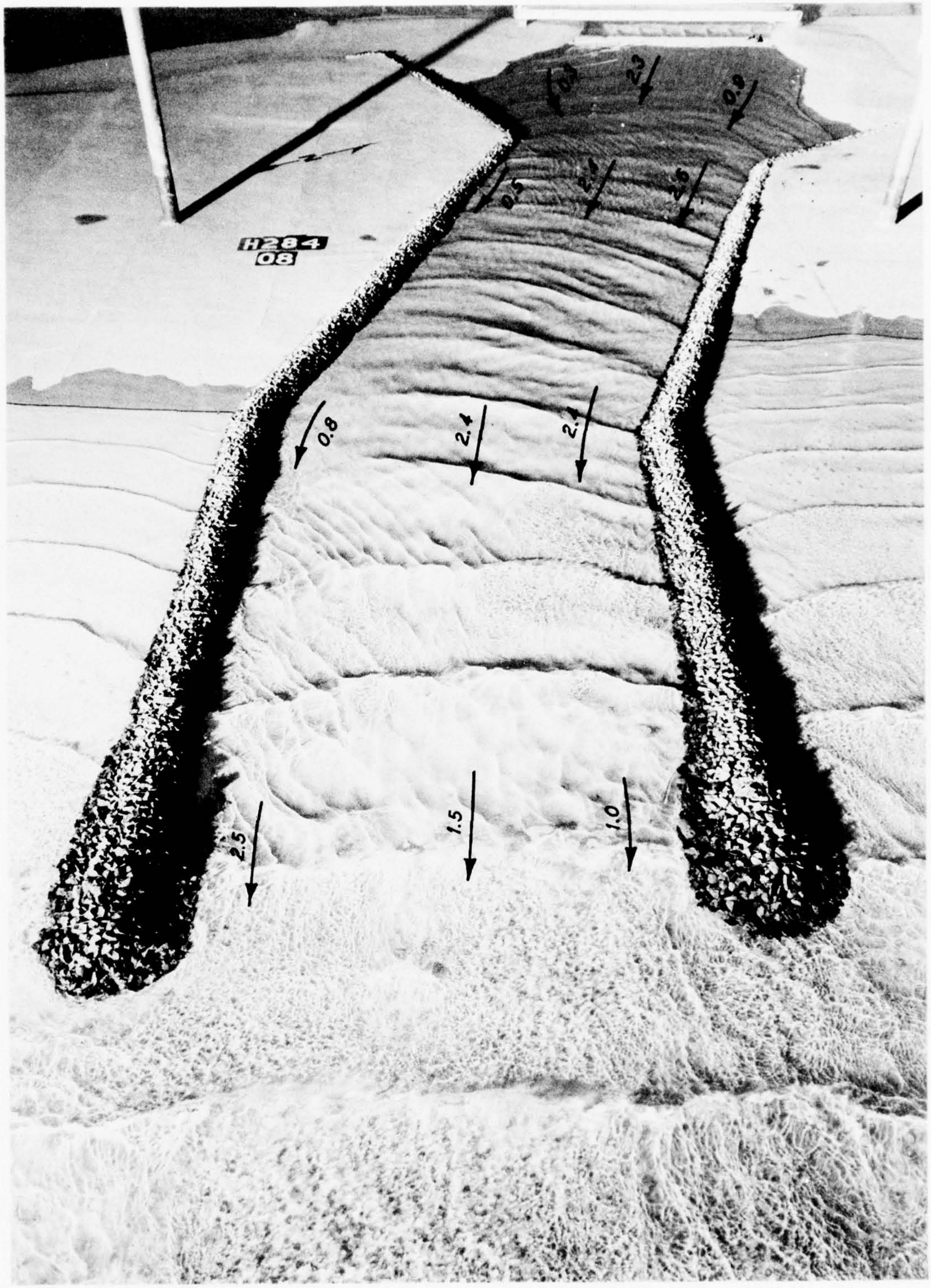


Photo 2. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 8-sec, 10-ft waves for maximum ebb

Photo 3. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 11-sec, 10-ft waves for maximum ebb

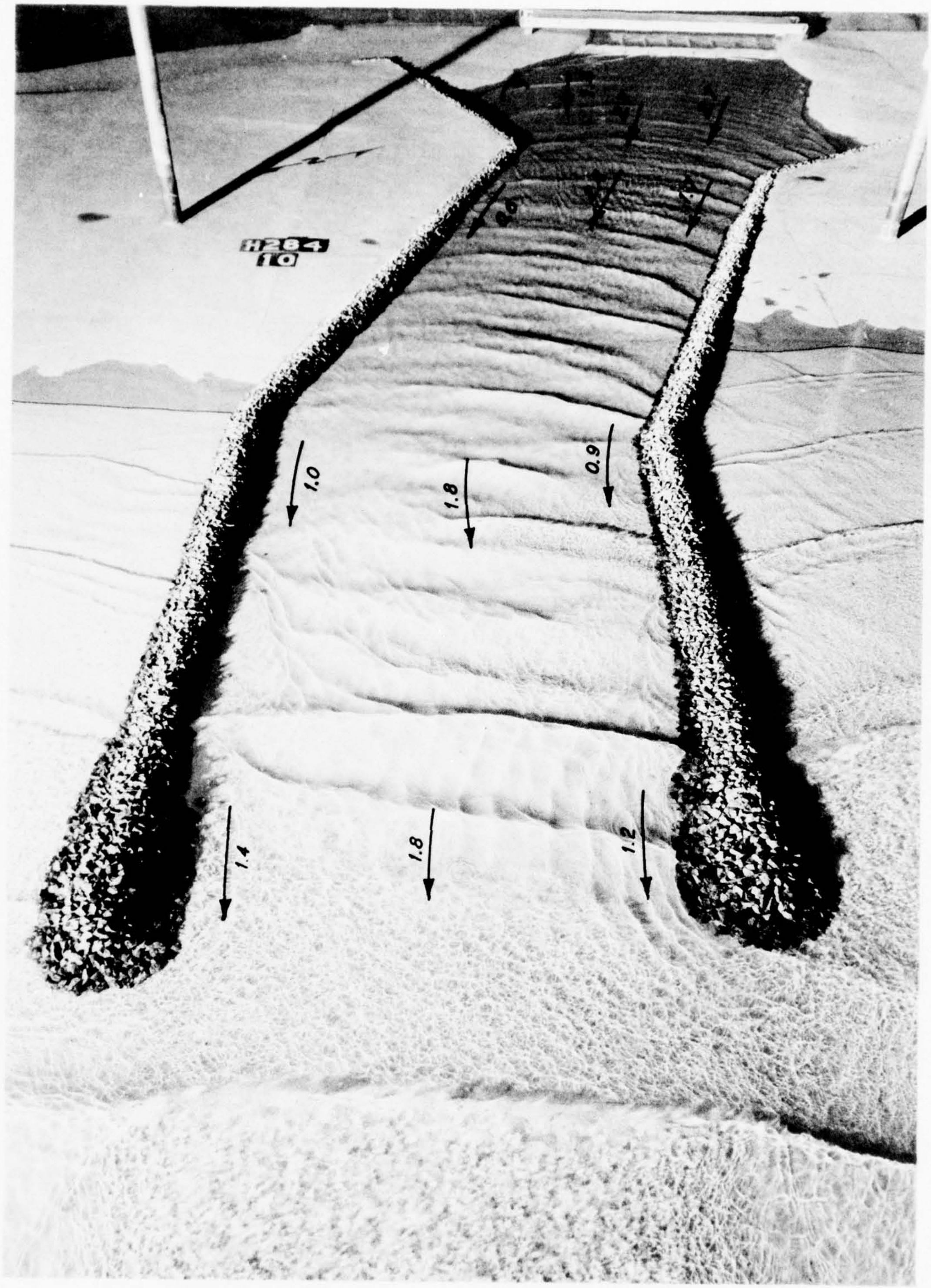


Photo 4. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 14-sec, 7-ft waves for maximum ebb

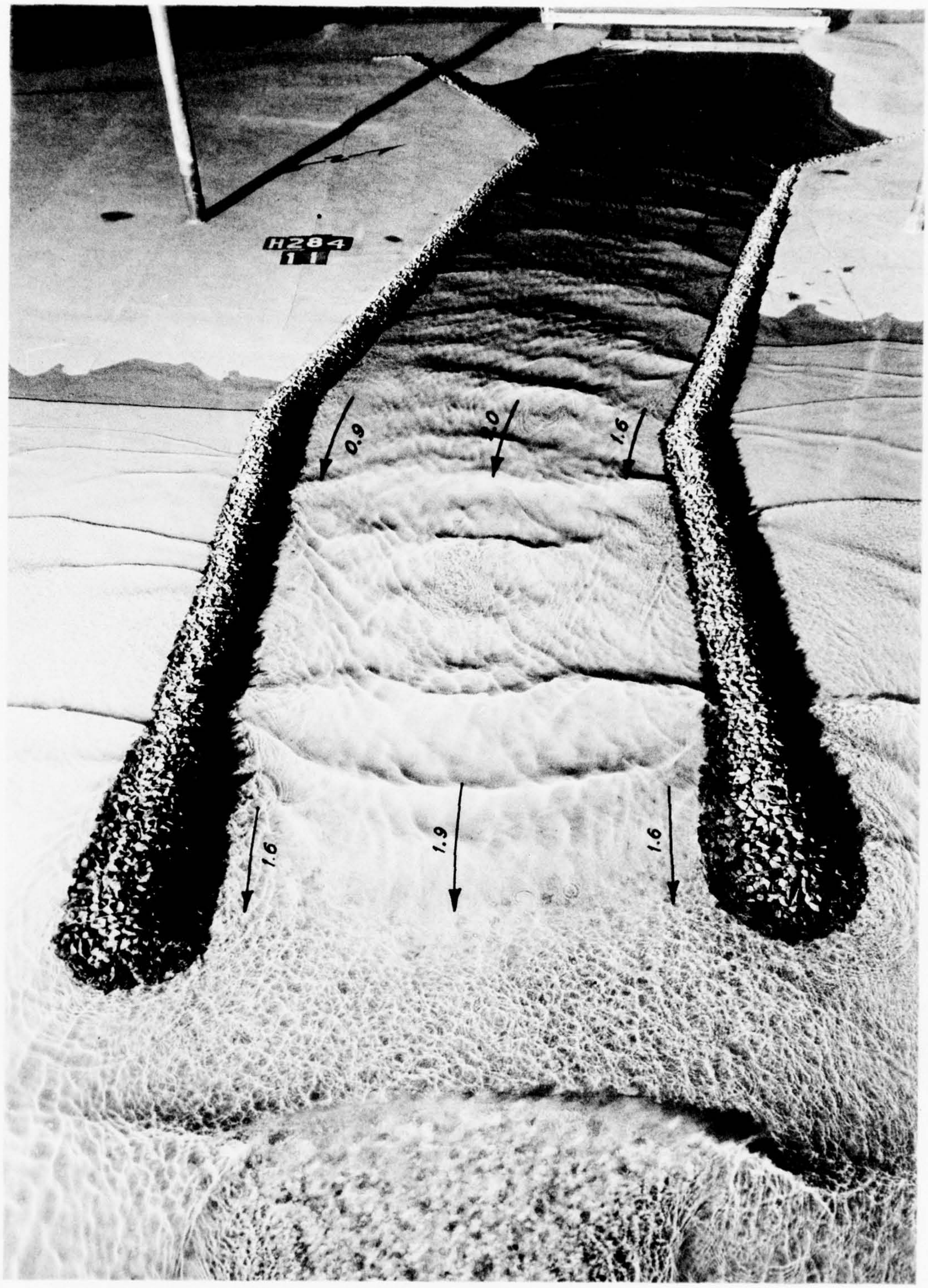


Photo 5. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 17-sec, 4-ft waves for maximum ebb

Photo 6. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 5-sec, 7-ft waves for maximum flood

Photo 7. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 8-sec, 10-ft waves for maximum flood

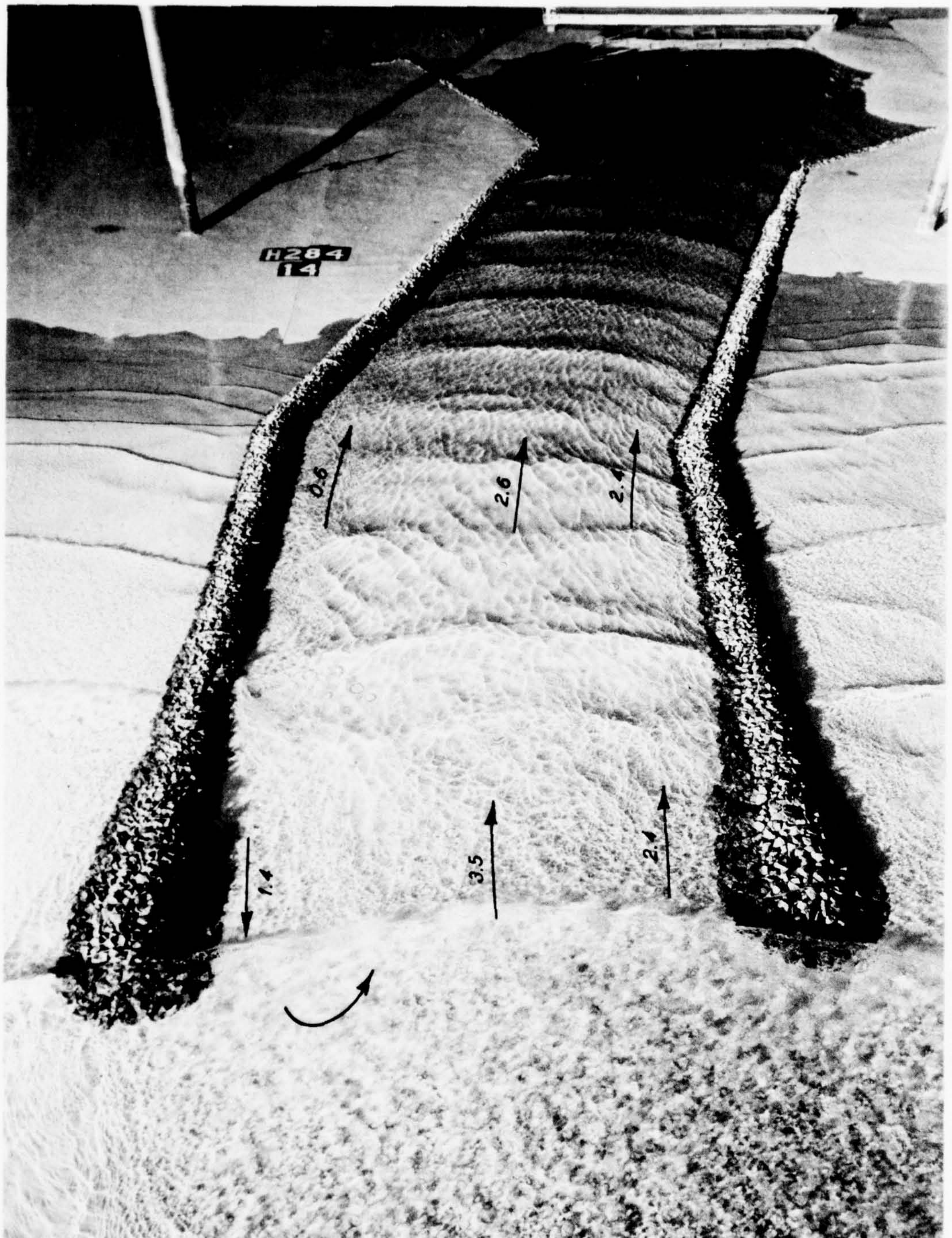


Photo 8. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 11-sec, 10-ft waves for maximum flood

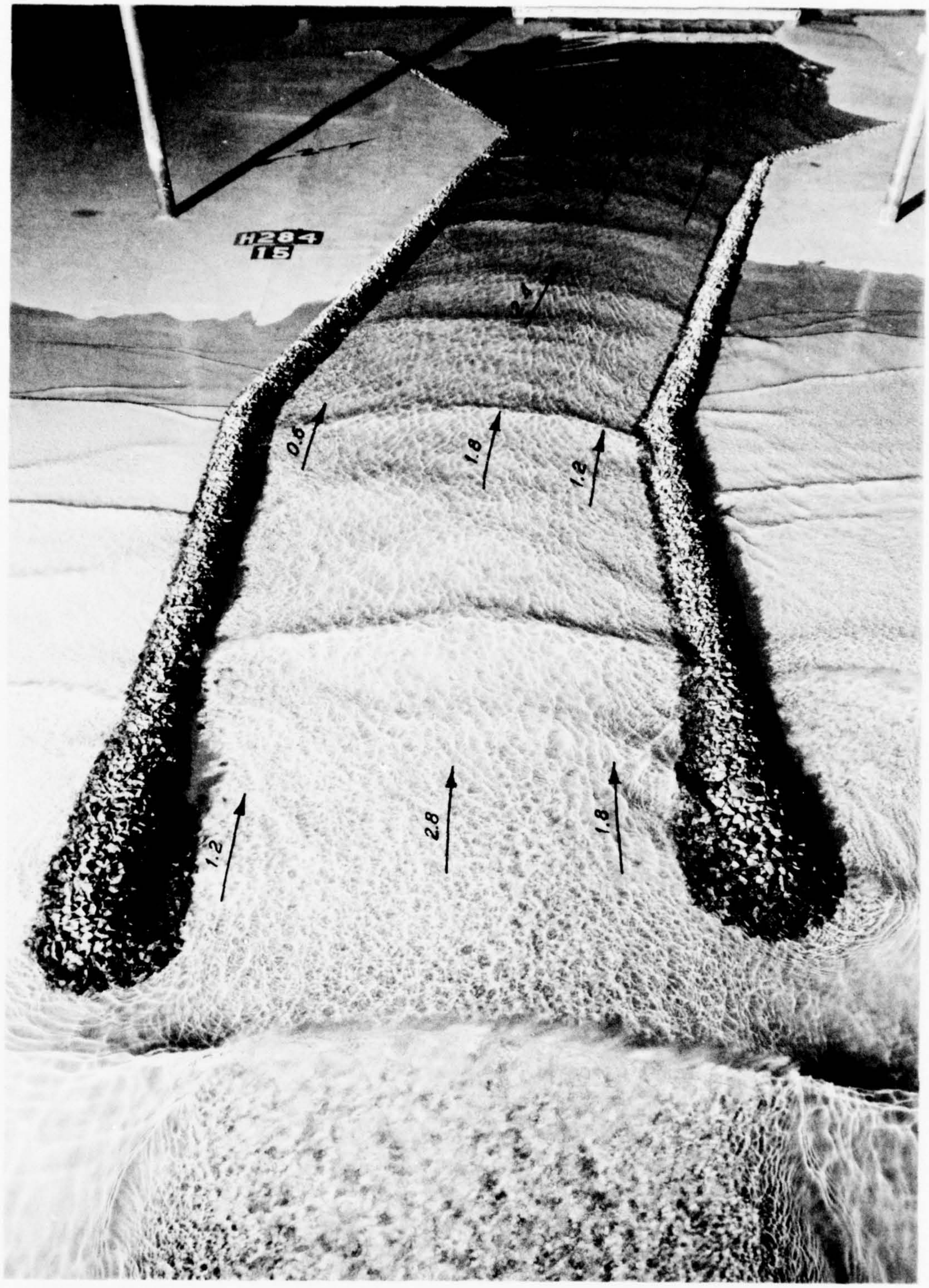


Photo 9. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 14-sec, 7-ft waves for maximum flood

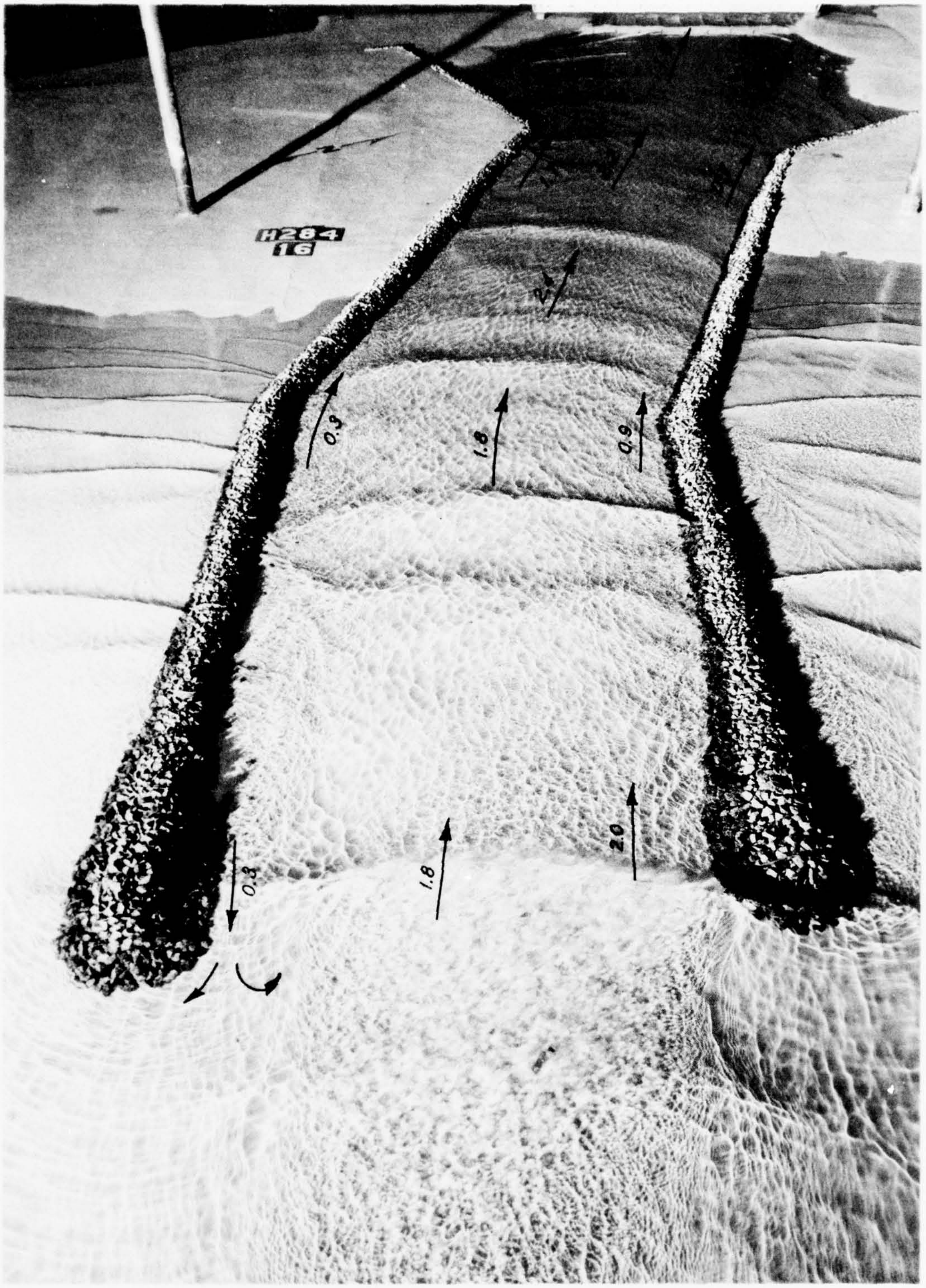


Photo 10. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for existing conditions; 17-sec, 4-ft waves for maximum flood

Photo 11. Typical wave patterns for existing conditions;
5-sec, 7-ft waves for slack water

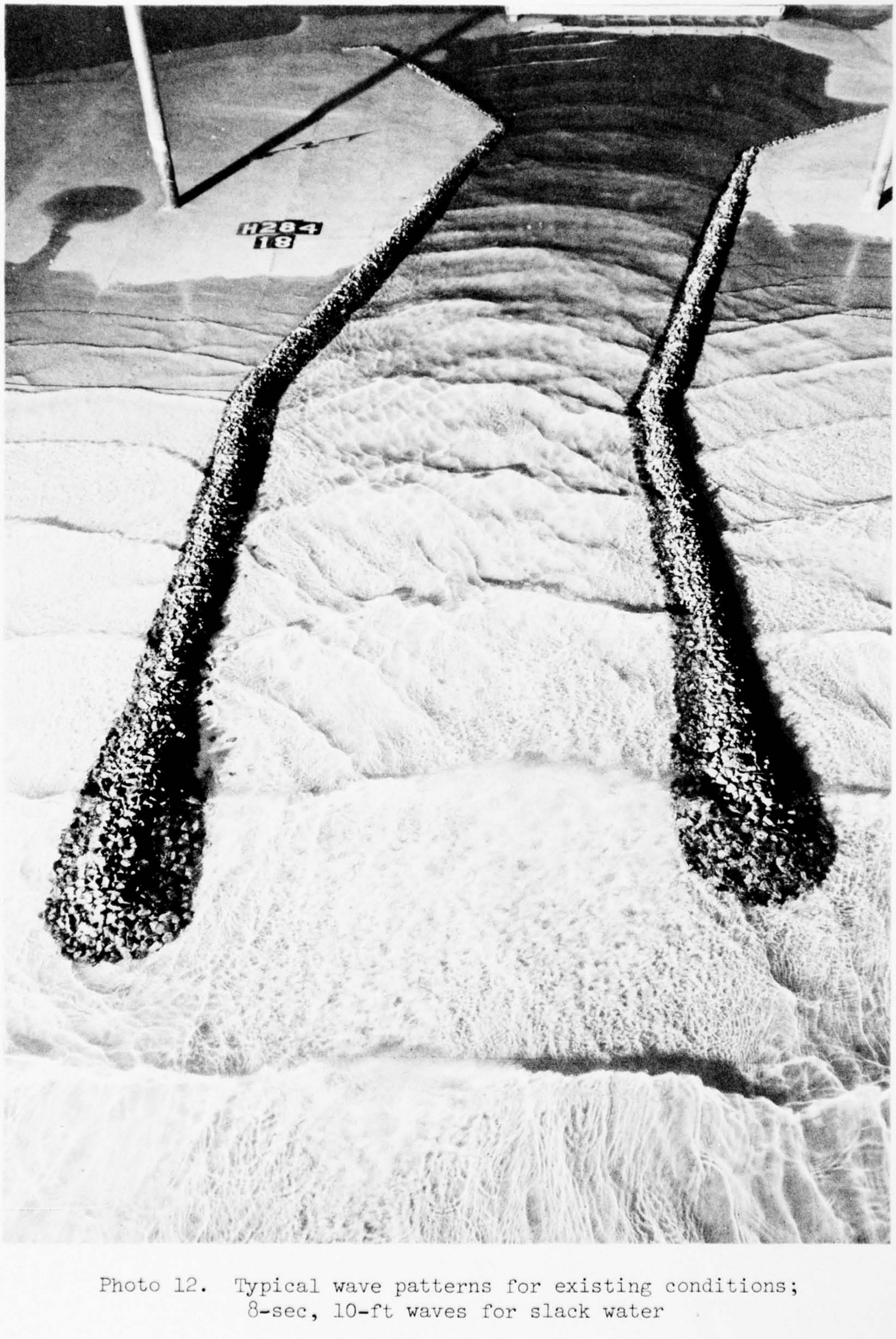


Photo 12. Typical wave patterns for existing conditions;
8-sec, 10-ft waves for slack water

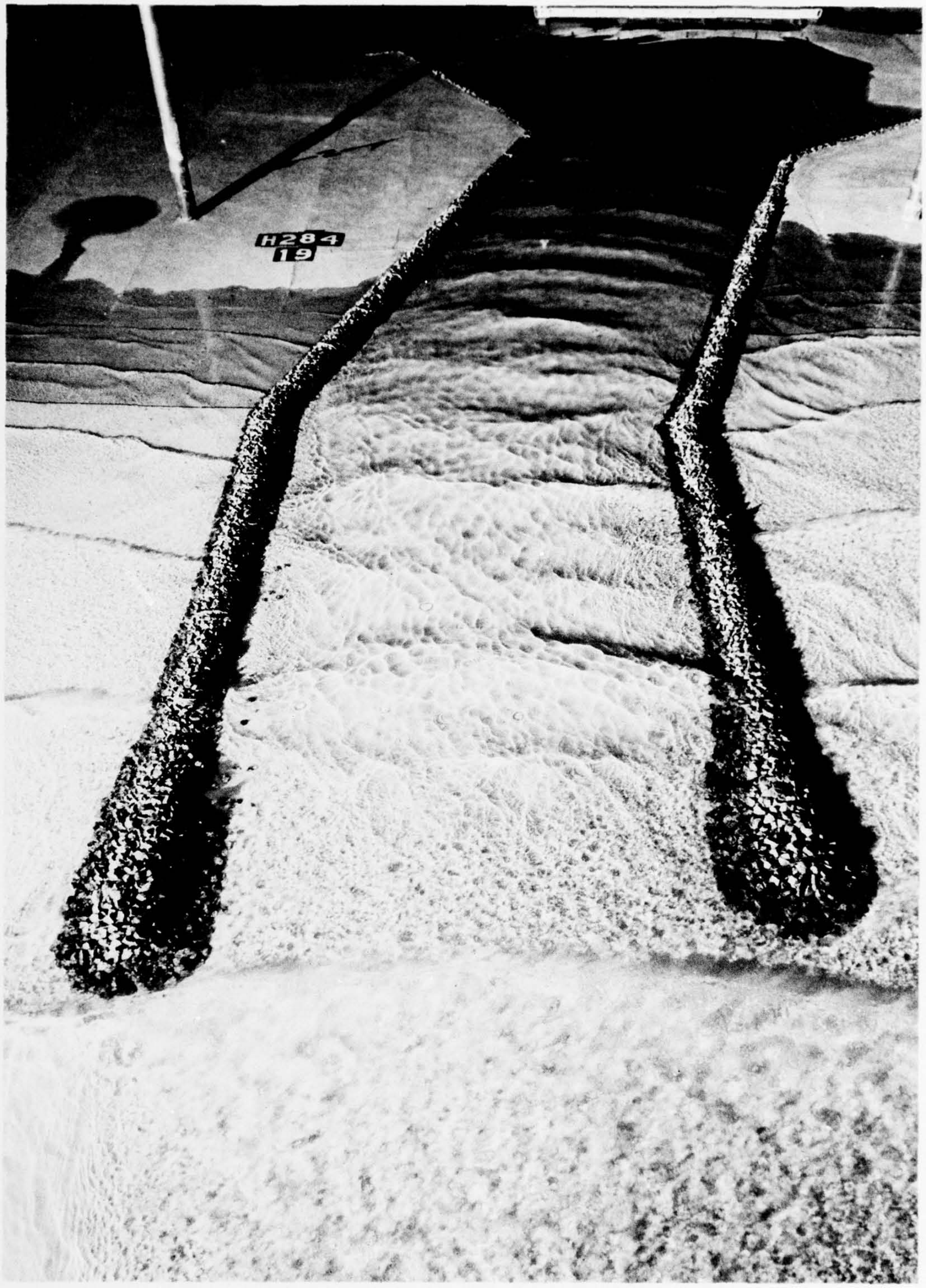


Photo 13. Typical wave patterns for existing conditions; 11-sec, 10-ft waves for slack water

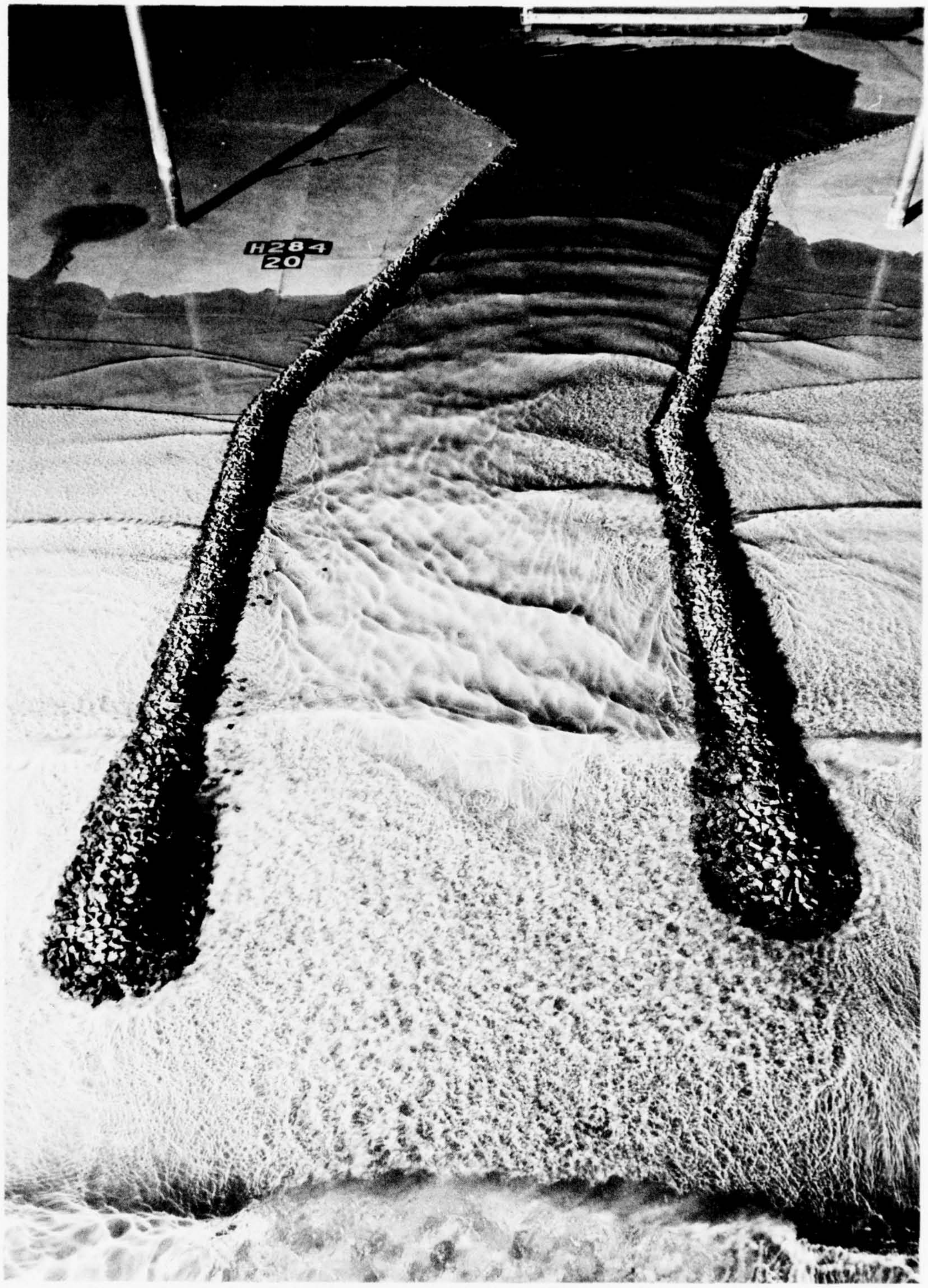


Photo 14. Typical wave patterns for existing conditions; 14-sec, 7-ft waves for slack water

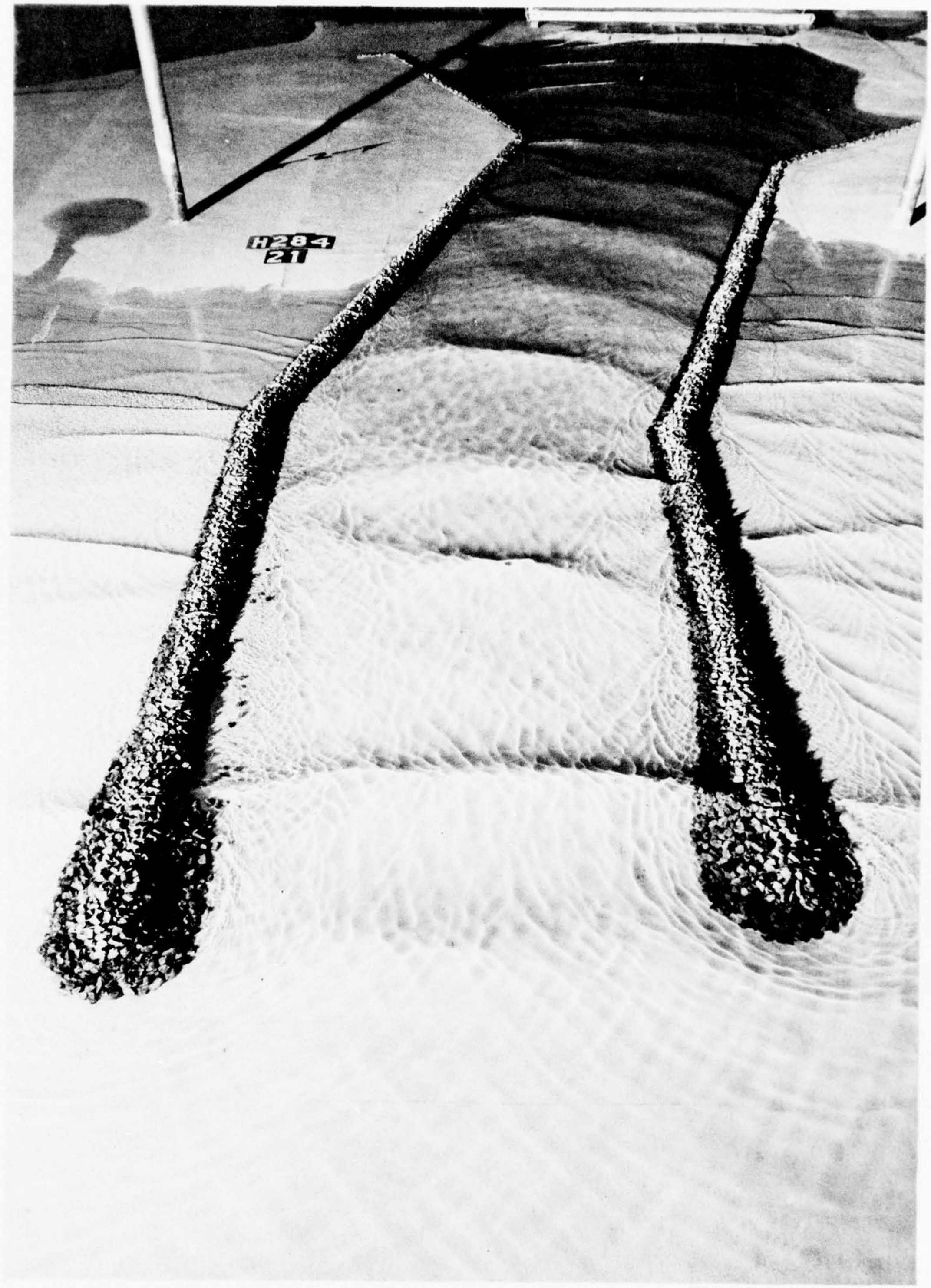


Photo 15. Typical wave patterns for existing conditions; 17-sec, 4-ft waves for slack water

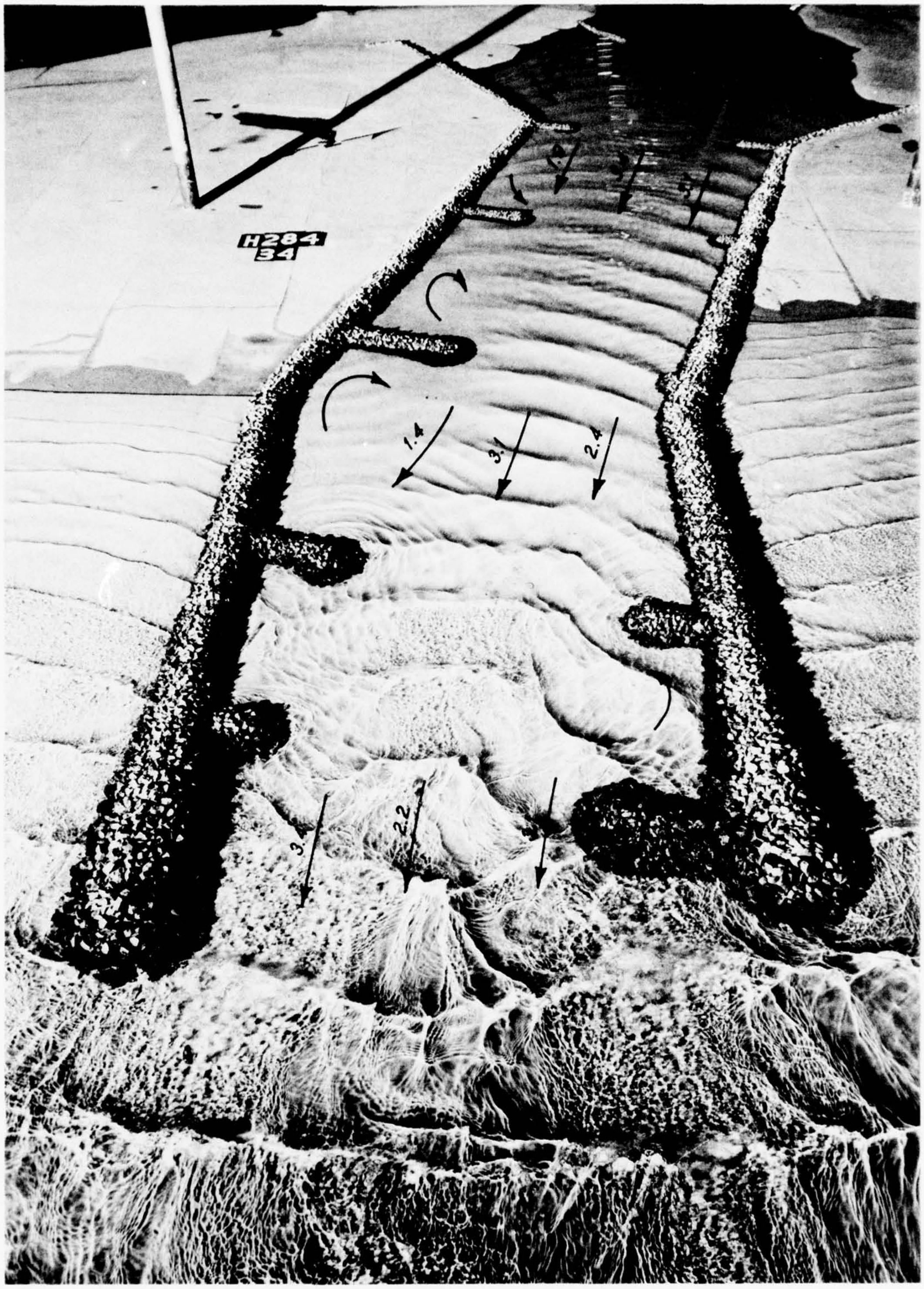


Photo 16. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 1; 5-sec, 7-ft waves for maximum ebb

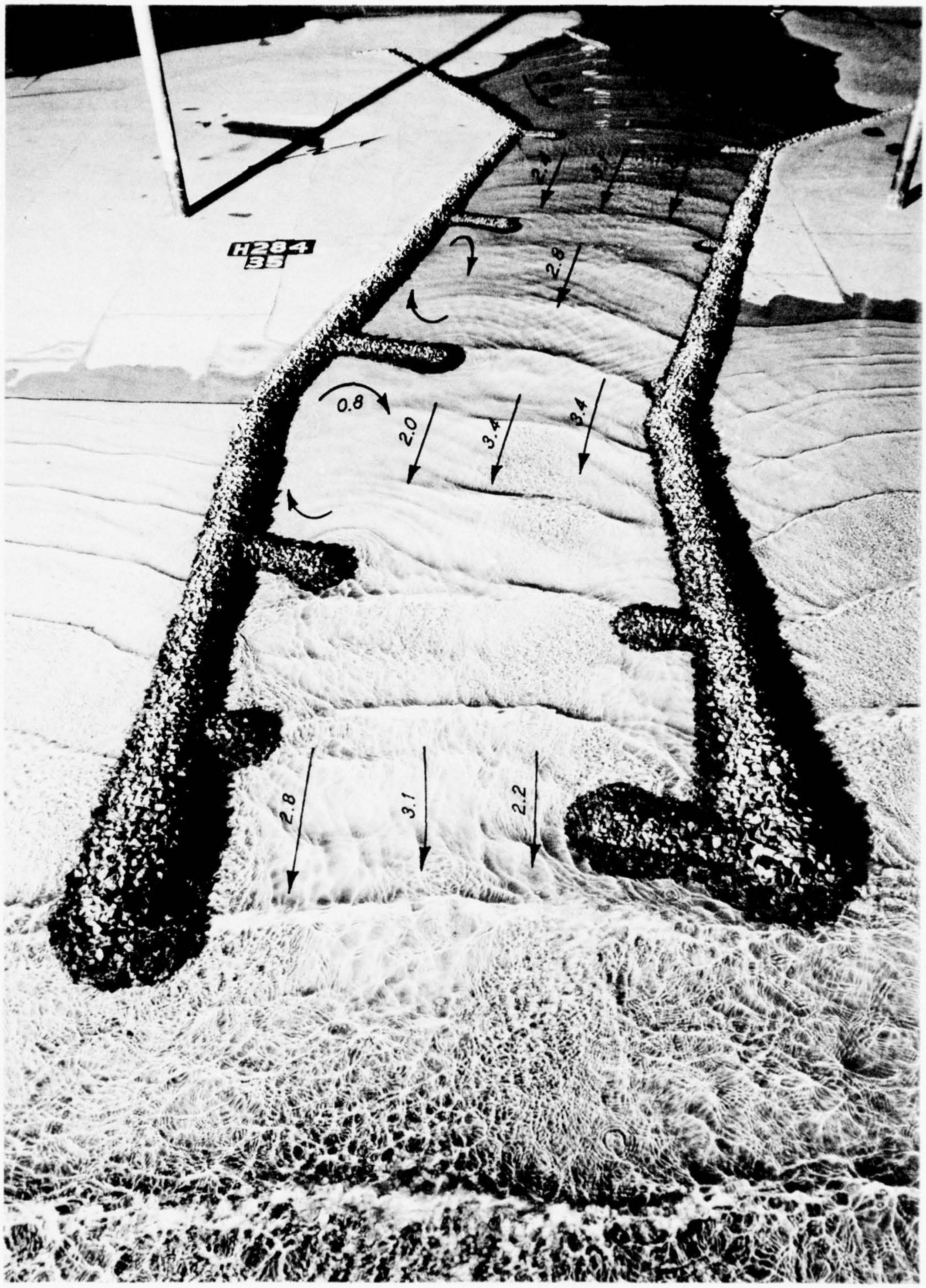


Photo 17. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 1; 8-sec, 10-ft waves for maximum ebb

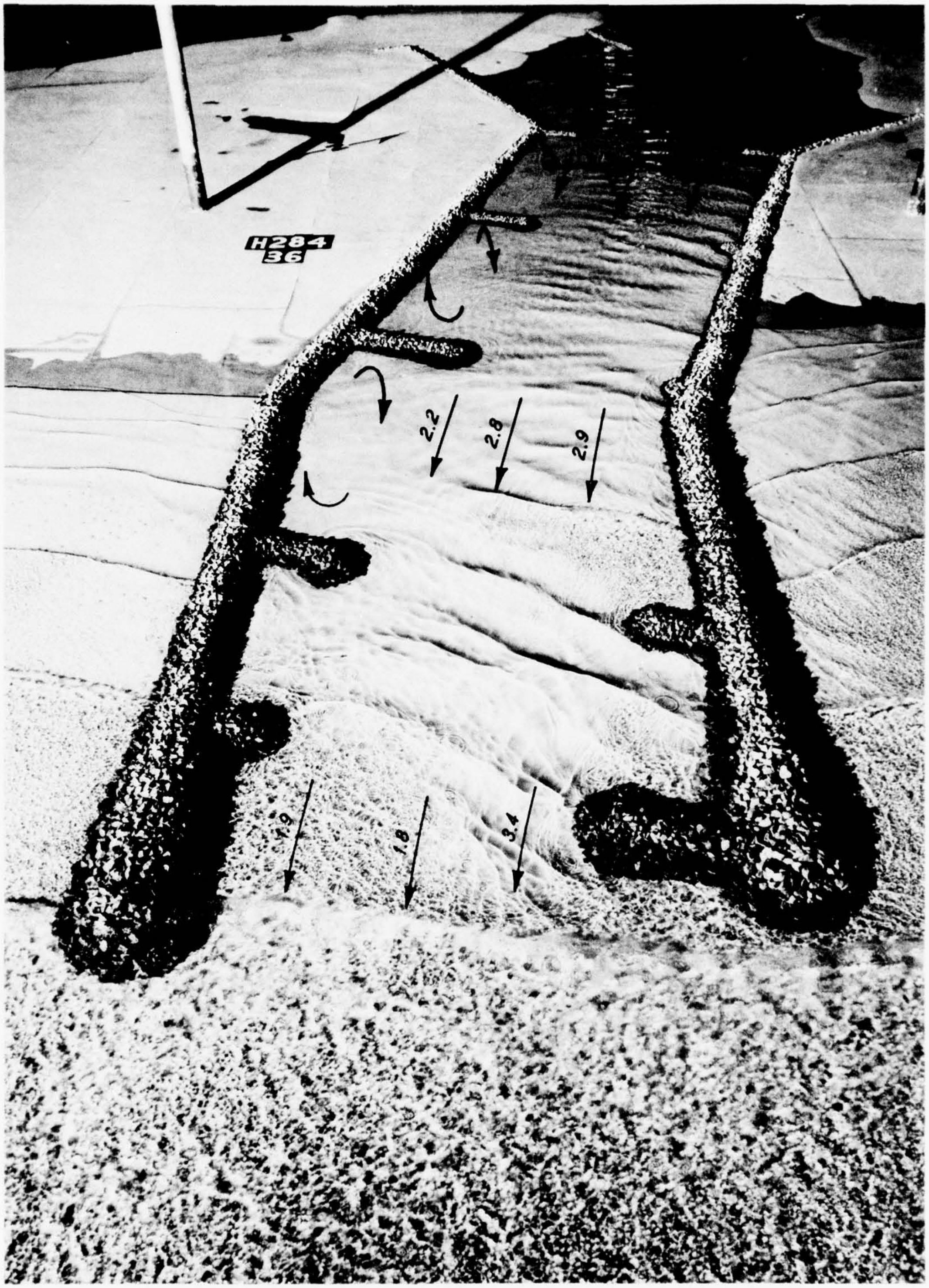


Photo 18. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 1; 11-sec, 10-ft waves for maximum ebb

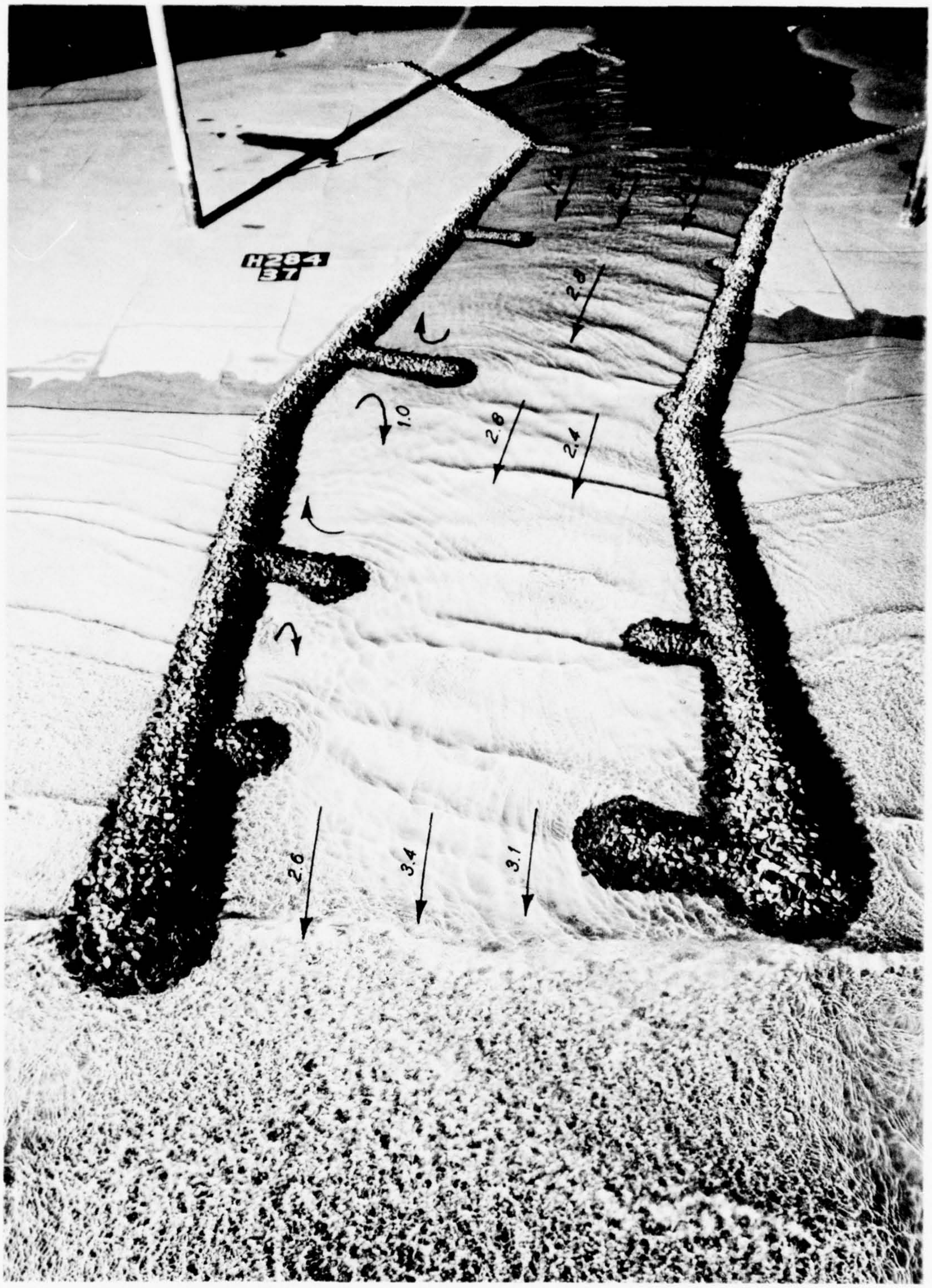


Photo 19. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 1; 14-sec, 7-ft waves for maximum ebb

Photo 20. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 1; 17-sec, 4-ft waves for maximum ebb

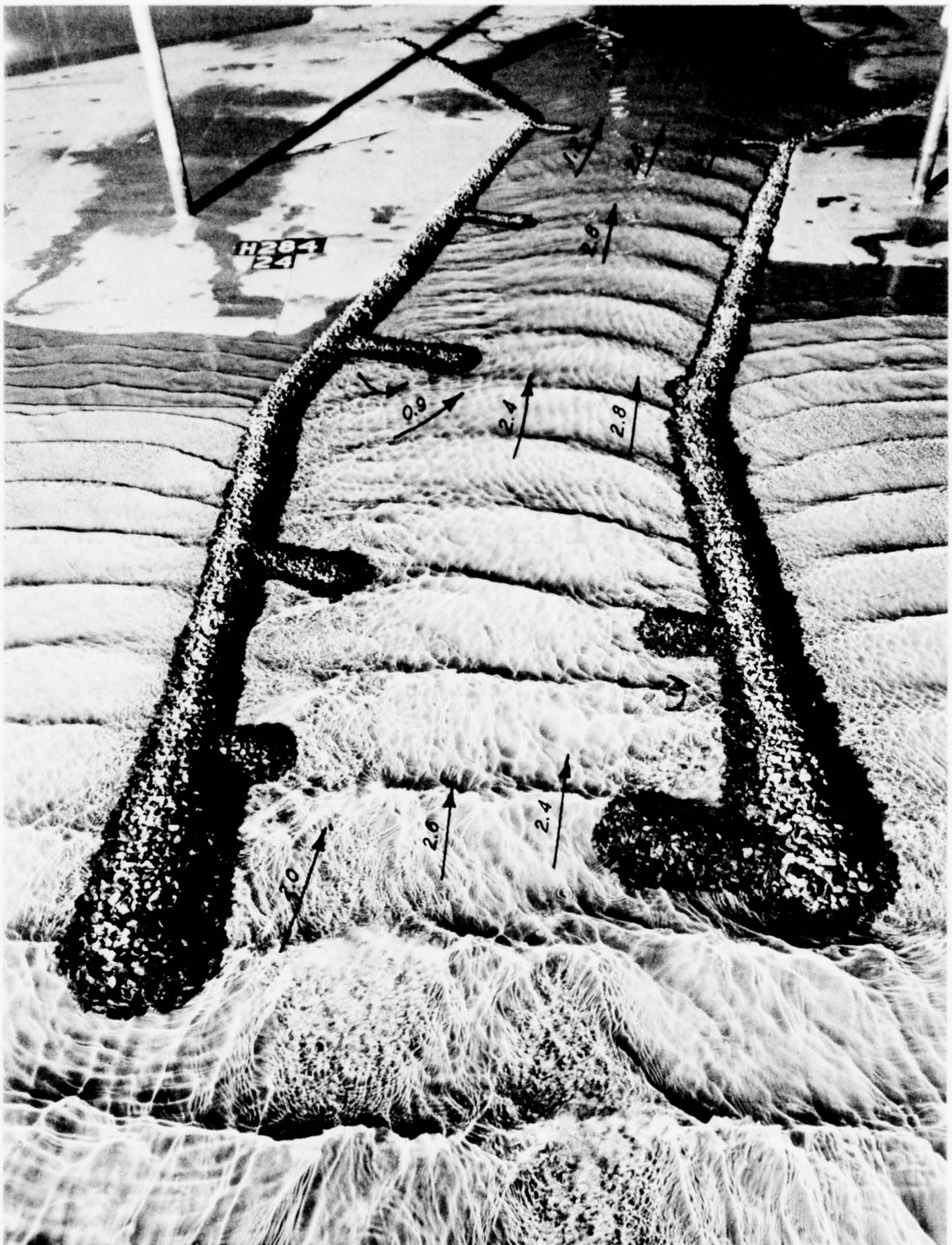


Photo 21. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 1; 5-sec, 7-ft waves for maximum flood

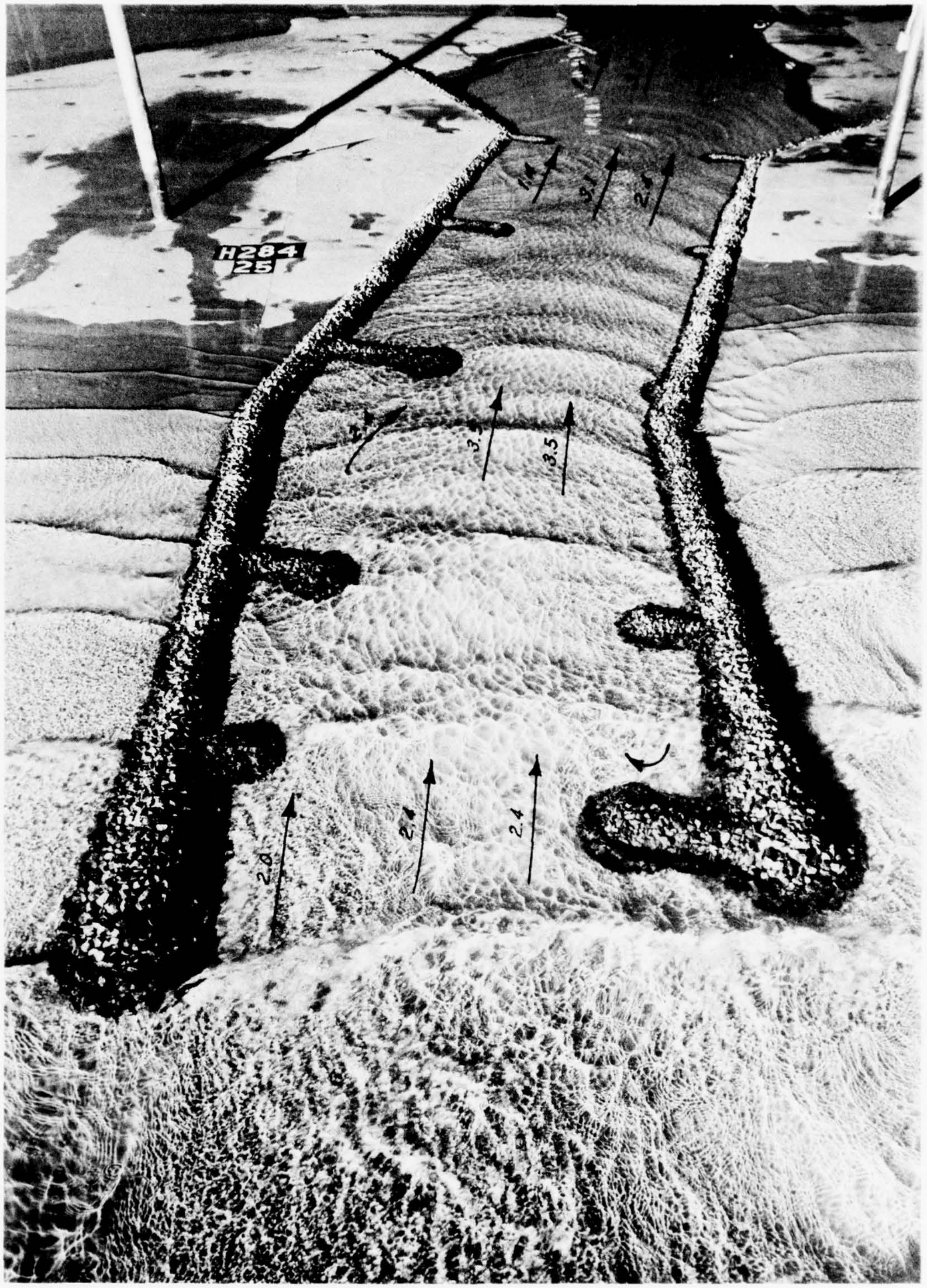


Photo 22. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 1; 8-sec, 10-ft waves for maximum flood

Photo 23. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 1; 11-sec, 10-ft waves for maximum flood

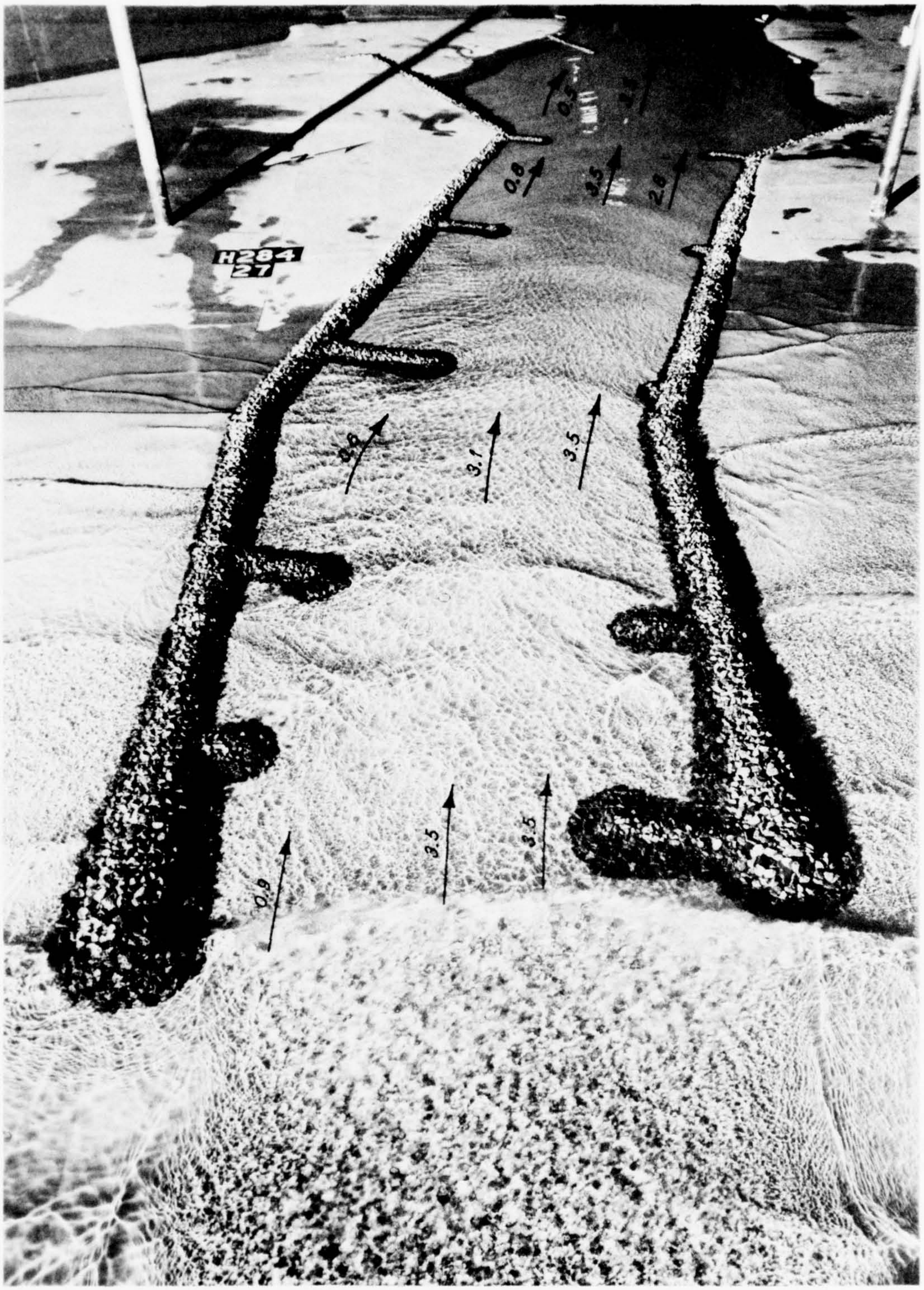


Photo 24. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 1; 14-sec, 7-ft waves for maximum flood

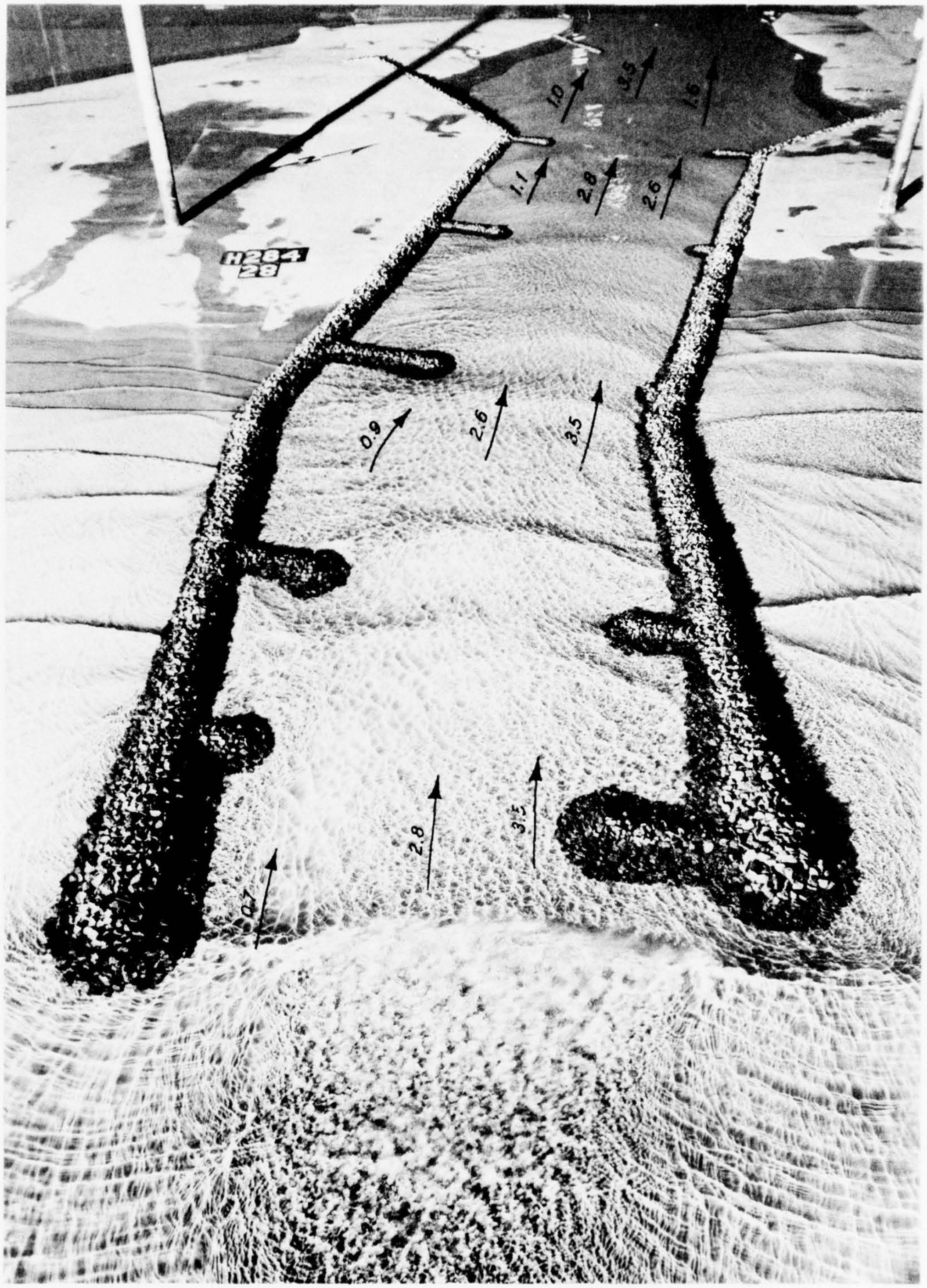


Photo 25. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 1; 17-sec, 4-ft waves for maximum flood

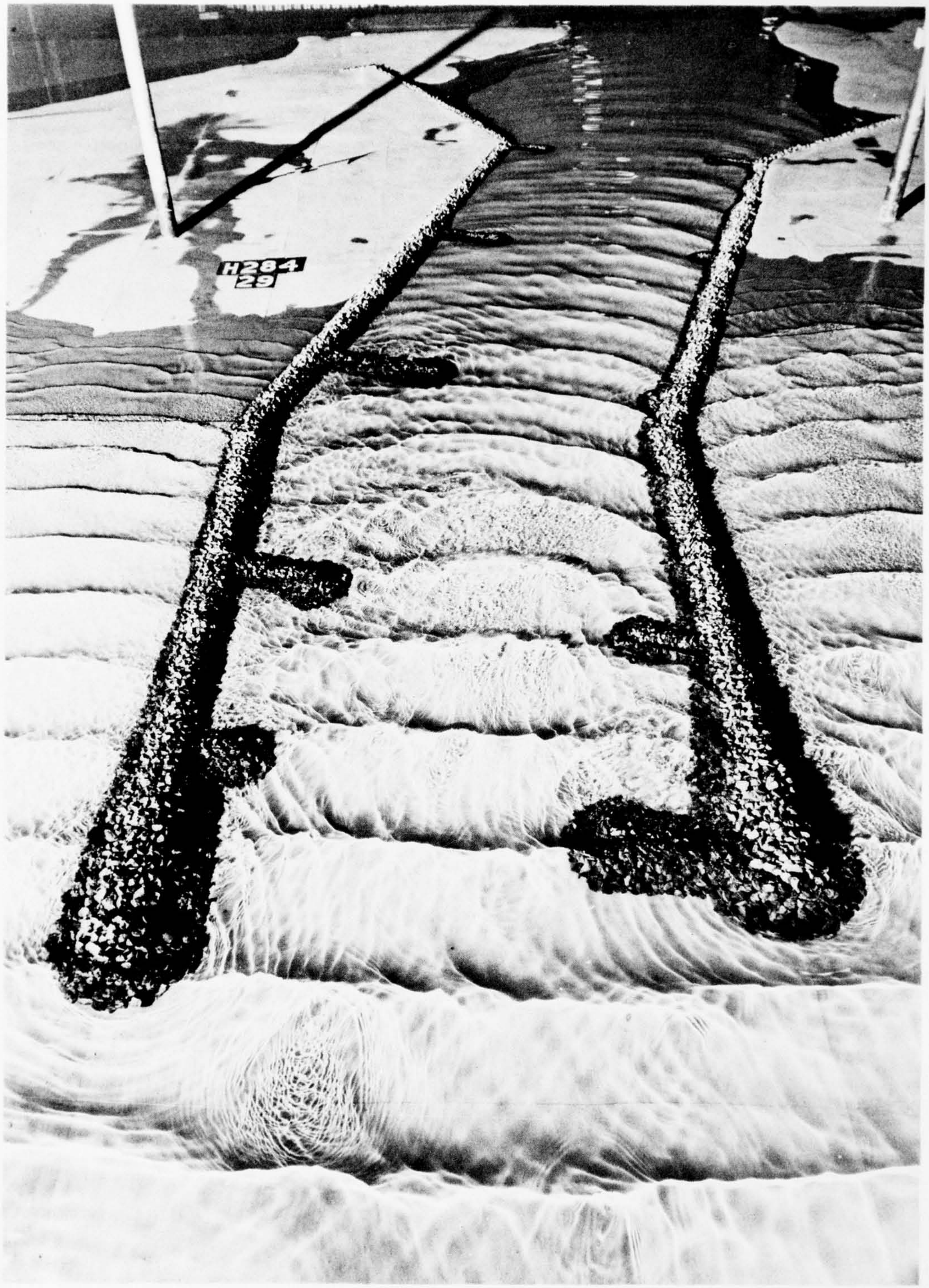


Photo 26. Typical wave patterns for plan 1;
5-sec, 7-ft waves for slack water

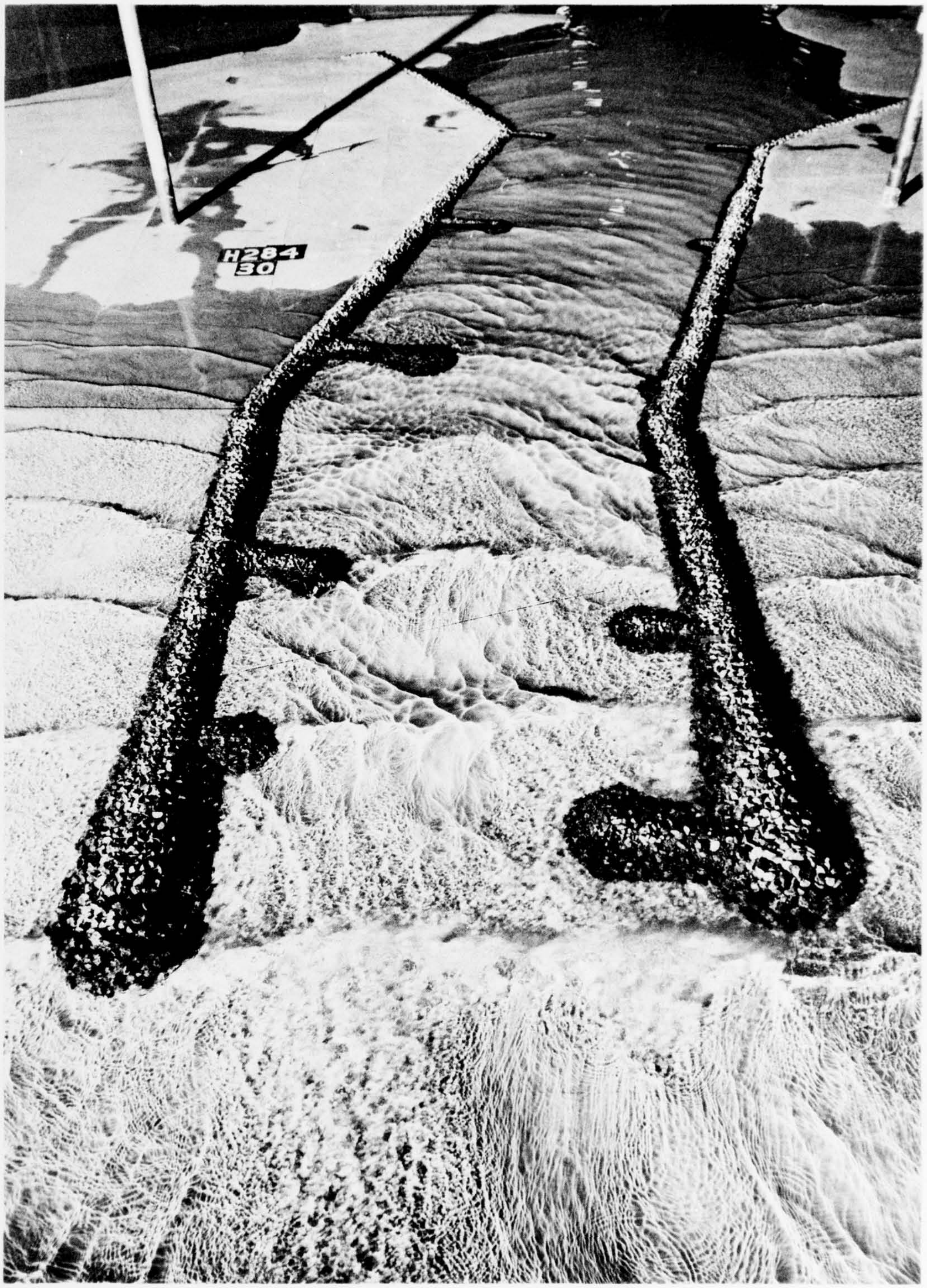


Photo 27. Typical wave patterns for plan 1;
8-sec, 10-ft waves for slack water

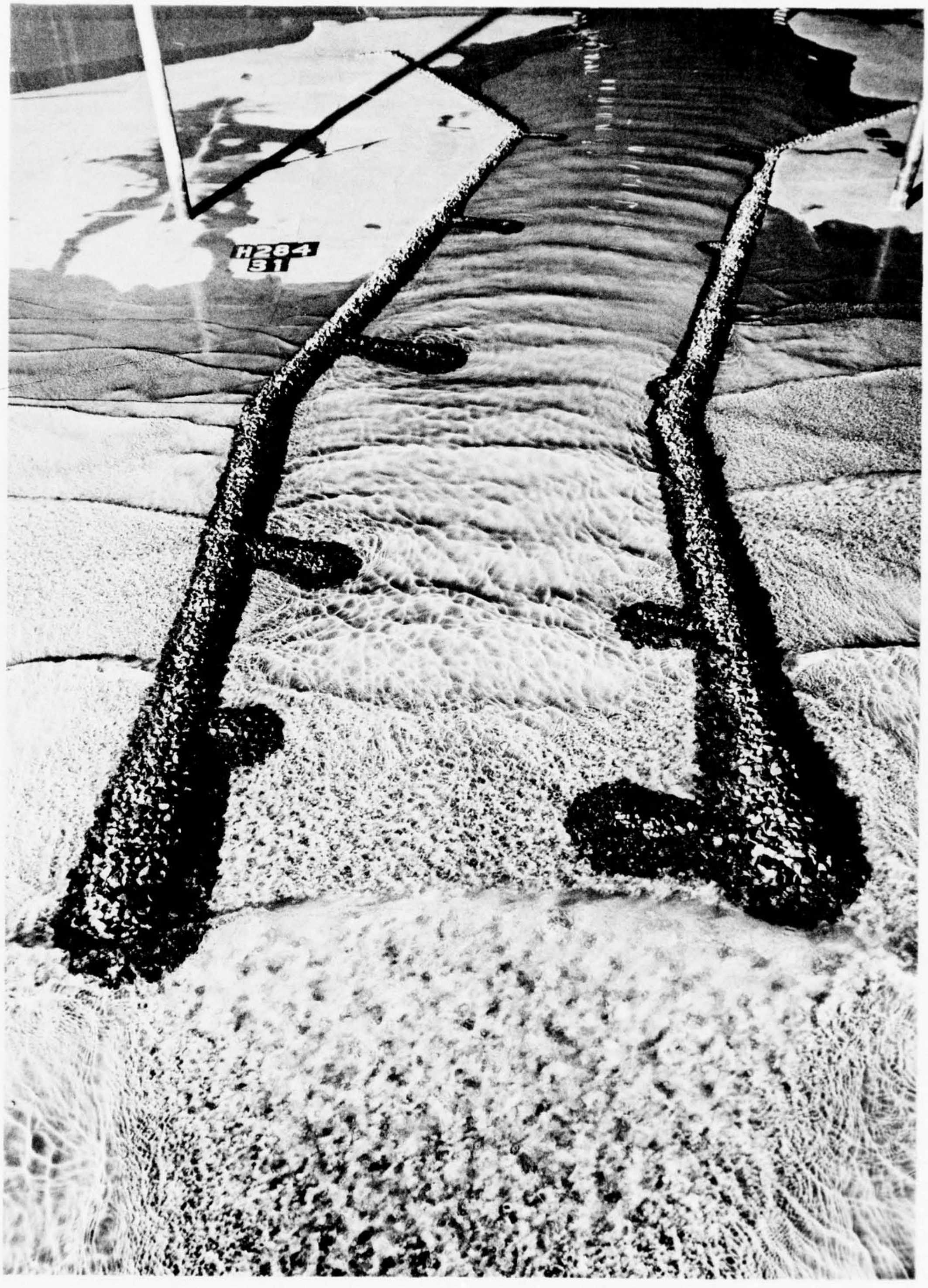


Photo 28. Typical wave patterns for plan 1;
11-sec, 10-ft waves for slack water

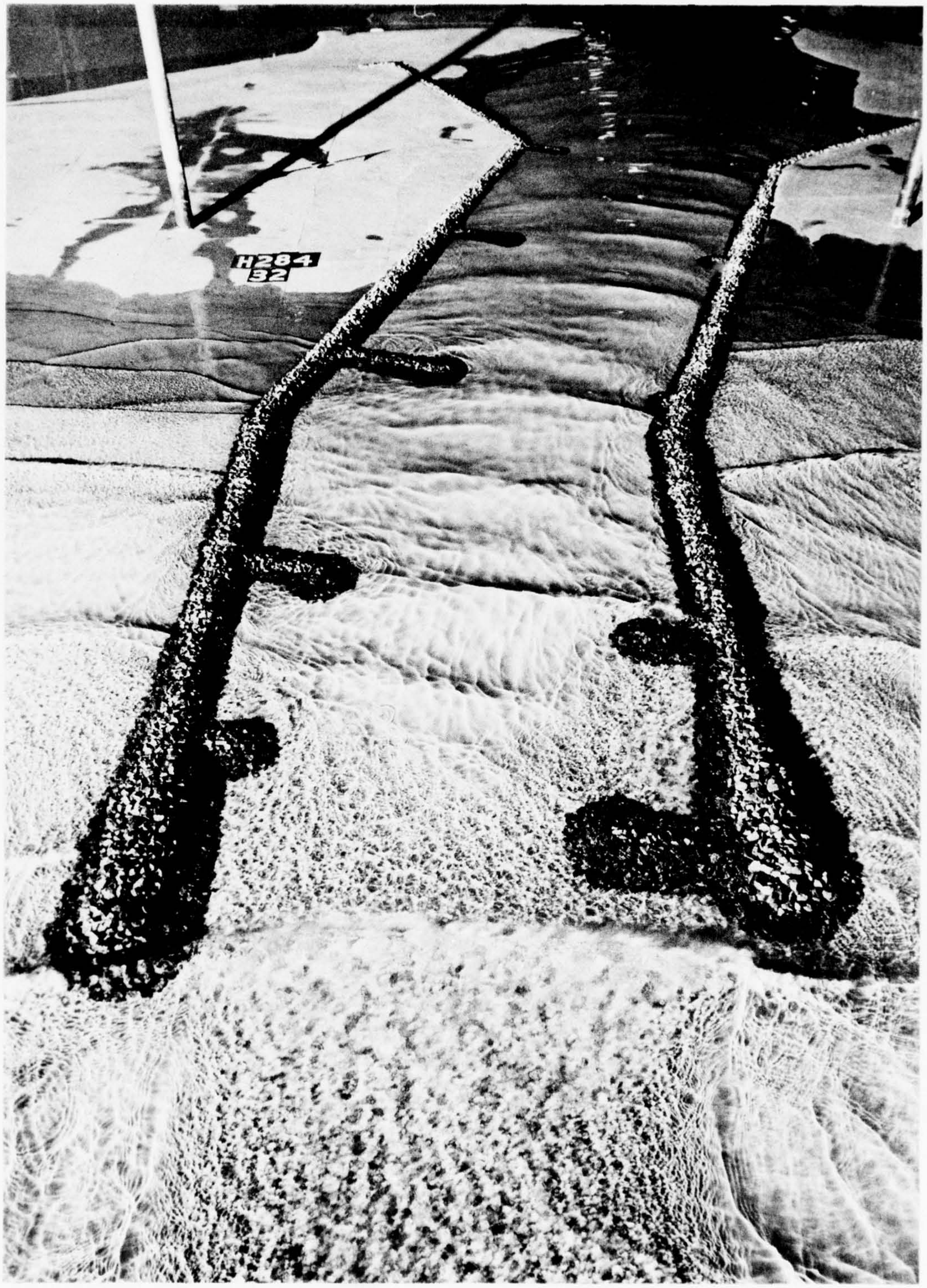


Photo 29. Typical wave patterns for plan 1;
14-sec, 7-ft waves for slack water

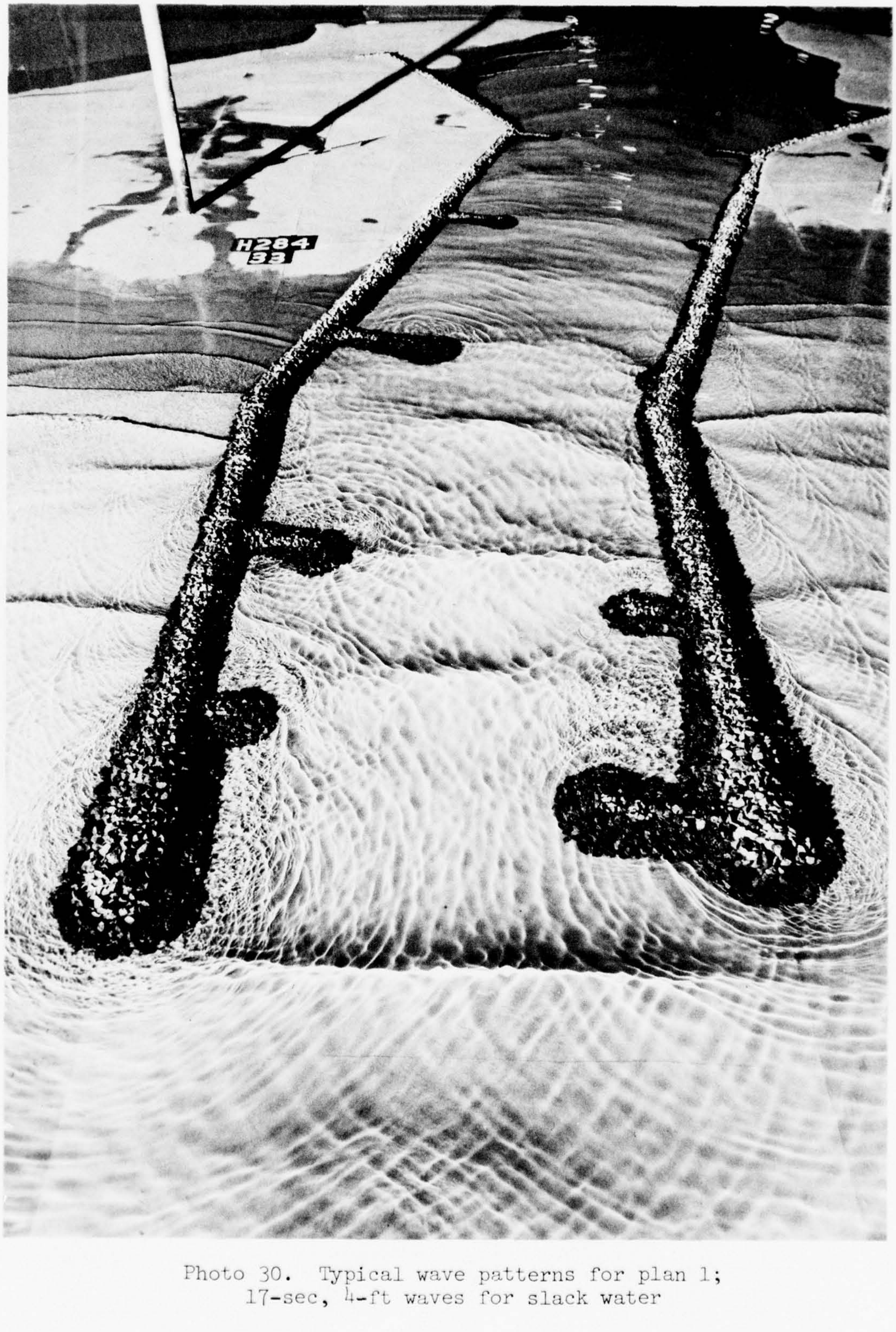


Photo 30. Typical wave patterns for plan 1;
17-sec, 4-ft waves for slack water

Photo 31. Typical wave patterns for plan 2;
5-sec, 7-ft waves for slack water

Photo 32. Typical wave patterns for plan 2;
11-sec, 10-ft waves for slack water

Photo 33. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 3; 5-sec, 7-ft waves for maximum ebb

Photo 34. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 3; 11-sec, 10-ft waves for maximum ebb

Photo 35. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 3; 17-sec, 4-ft waves for maximum ebb

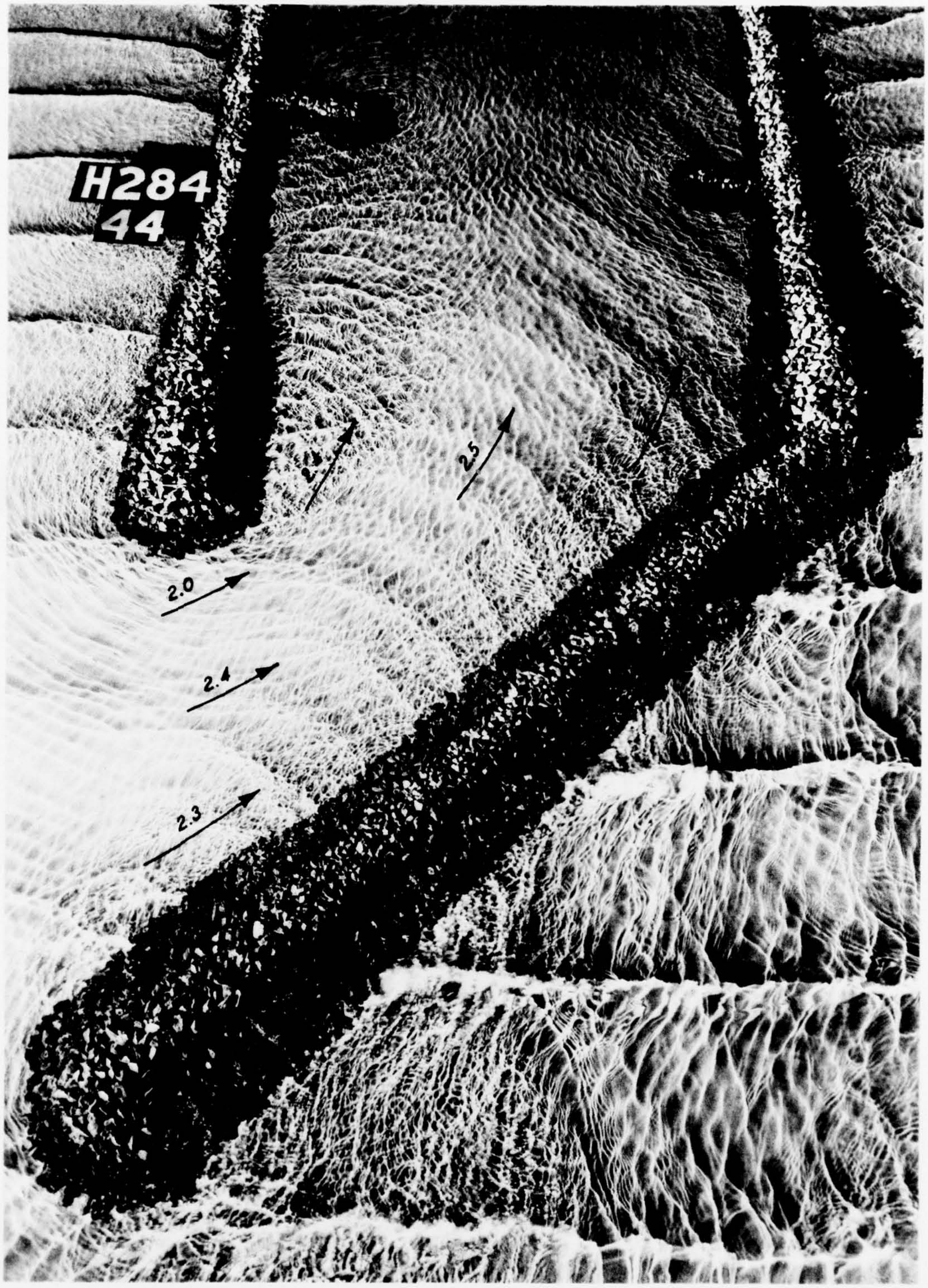


Photo 36. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 3; 5-sec, 7-ft waves for maximum flood

Photo 37. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 3; 11-sec, 10-ft waves for maximum flood

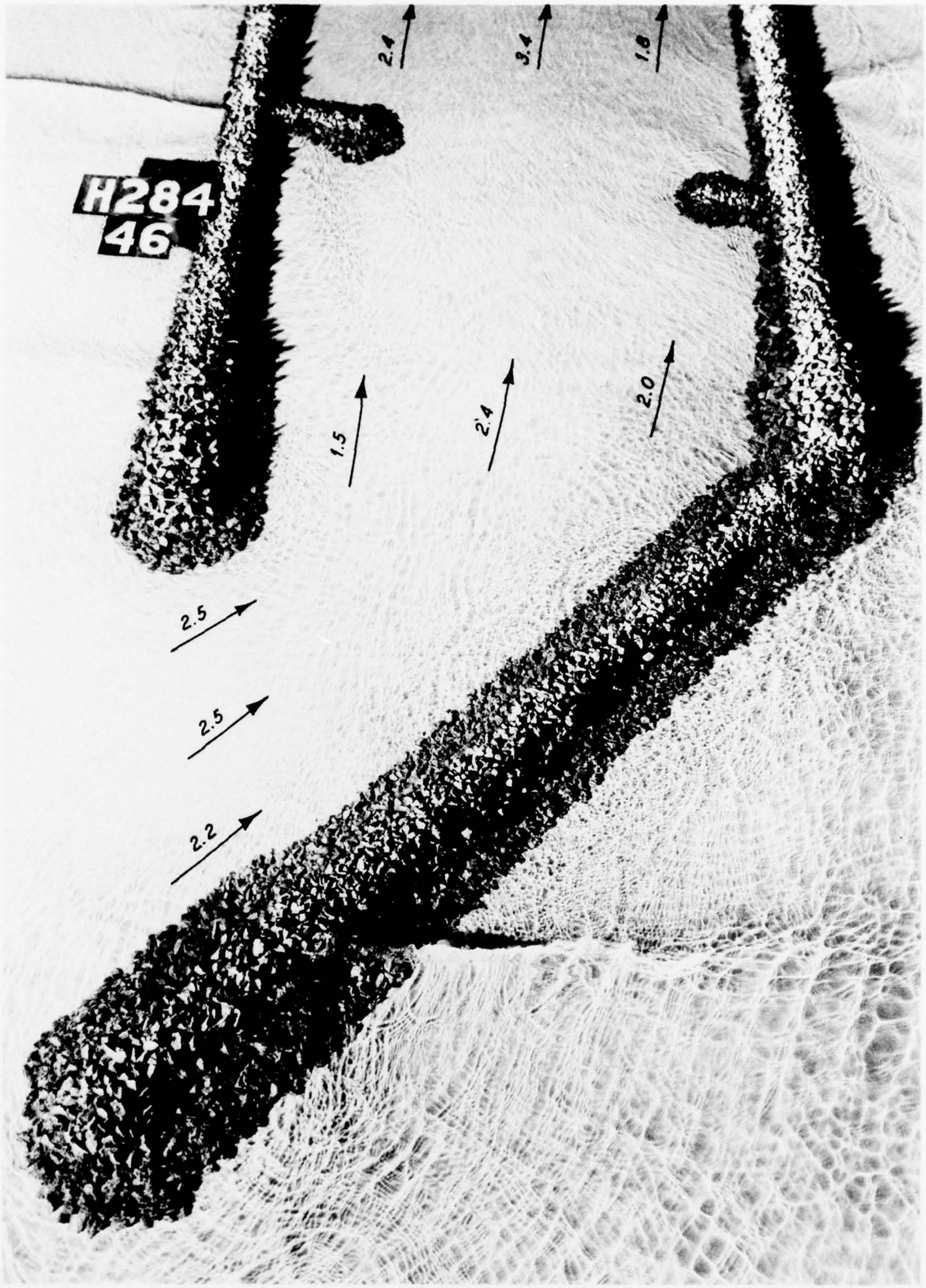


Photo 38. Typical wave patterns, current patterns, and current magnitudes (prototype feet per second) for plan 3; 17-sec, 4-ft waves for maximum flood

Photo 39. Typical wave patterns for plan 3;
5-sec, 7-ft waves for slack water

Photo 40. Typical wave patterns for plan 3;
11-sec, 10-ft waves for slack water

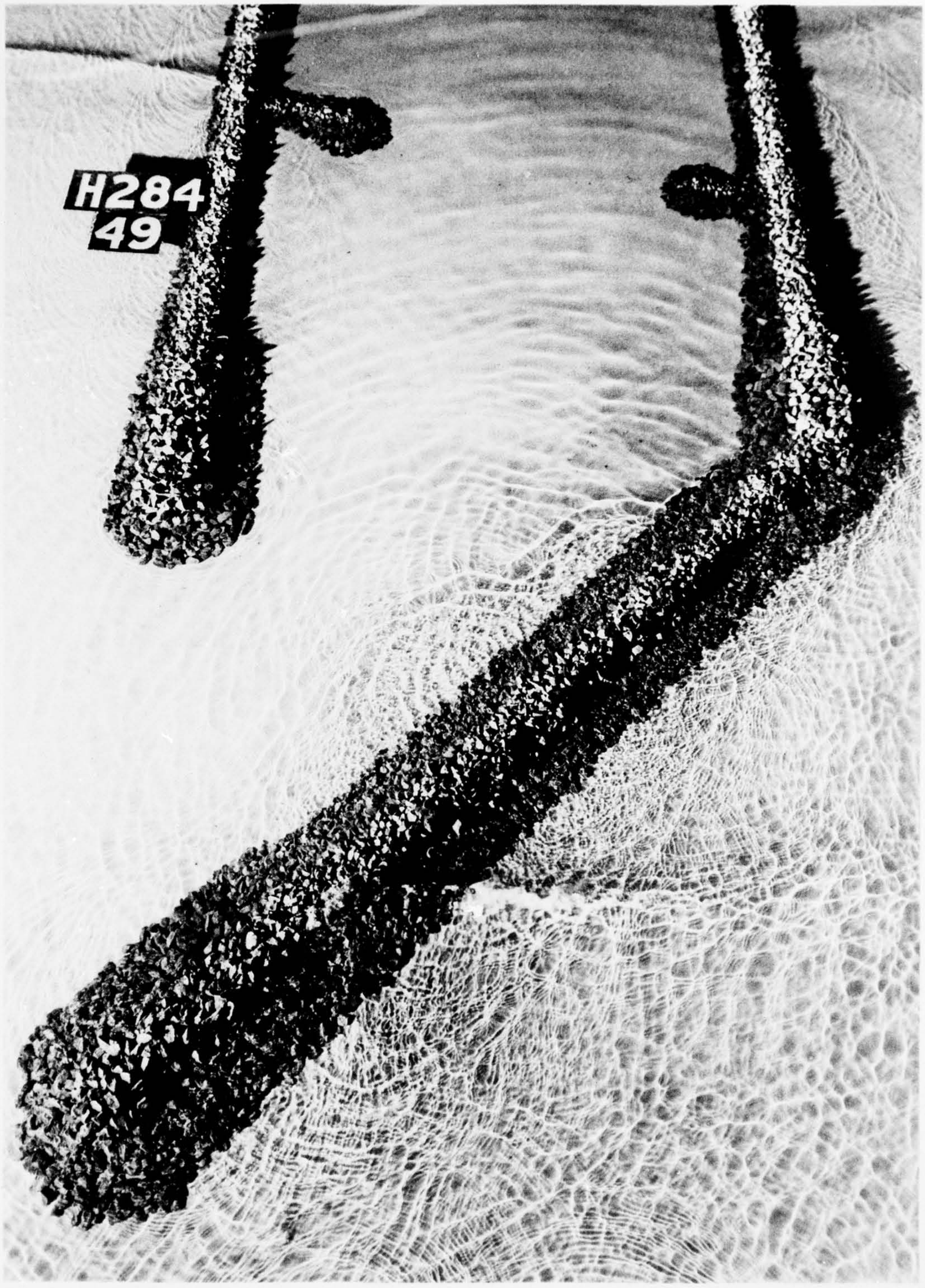
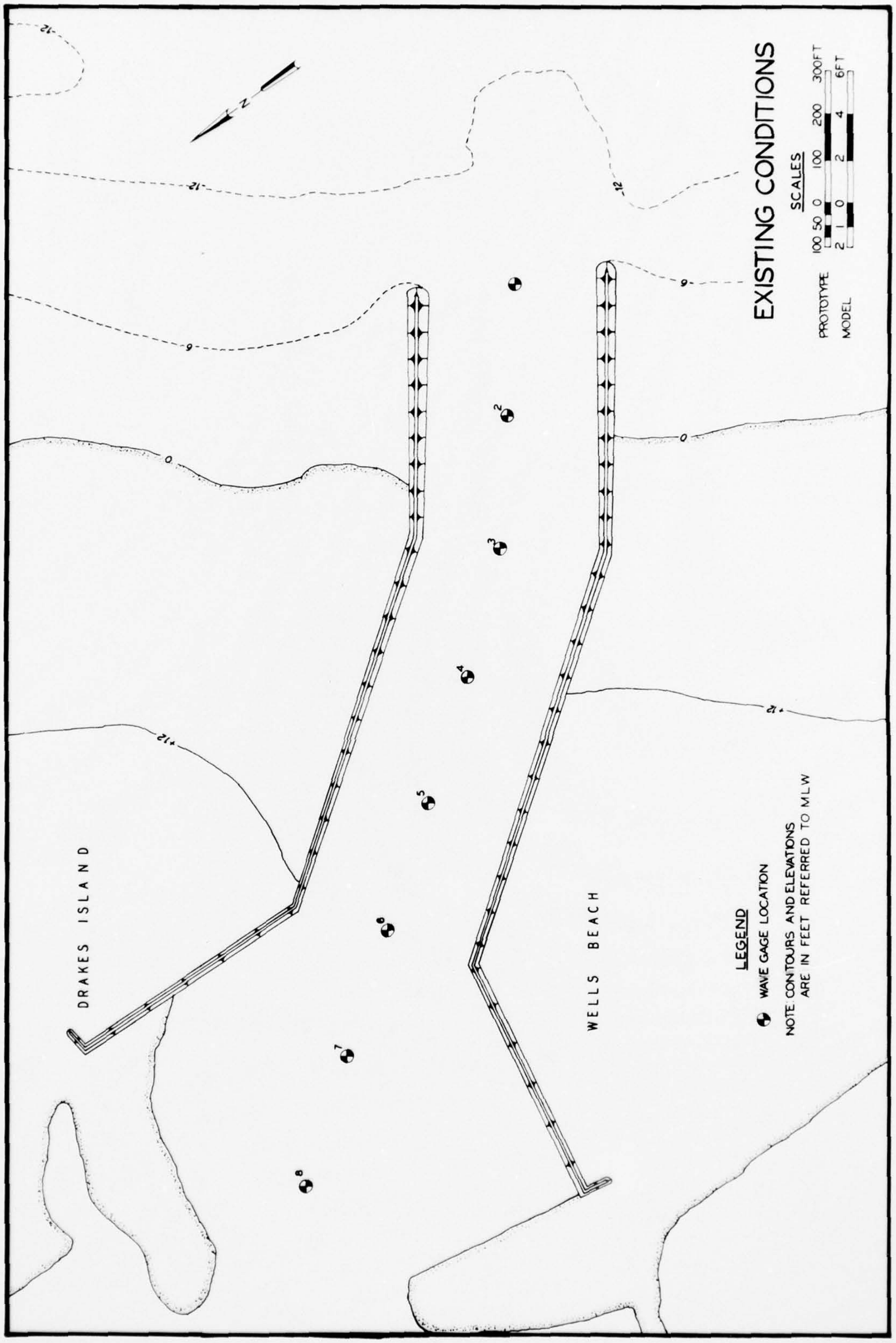


Photo 41. Typical wave patterns for plan 3;
17-sec, 4-ft waves for slack water


EXISTING CONDITIONS
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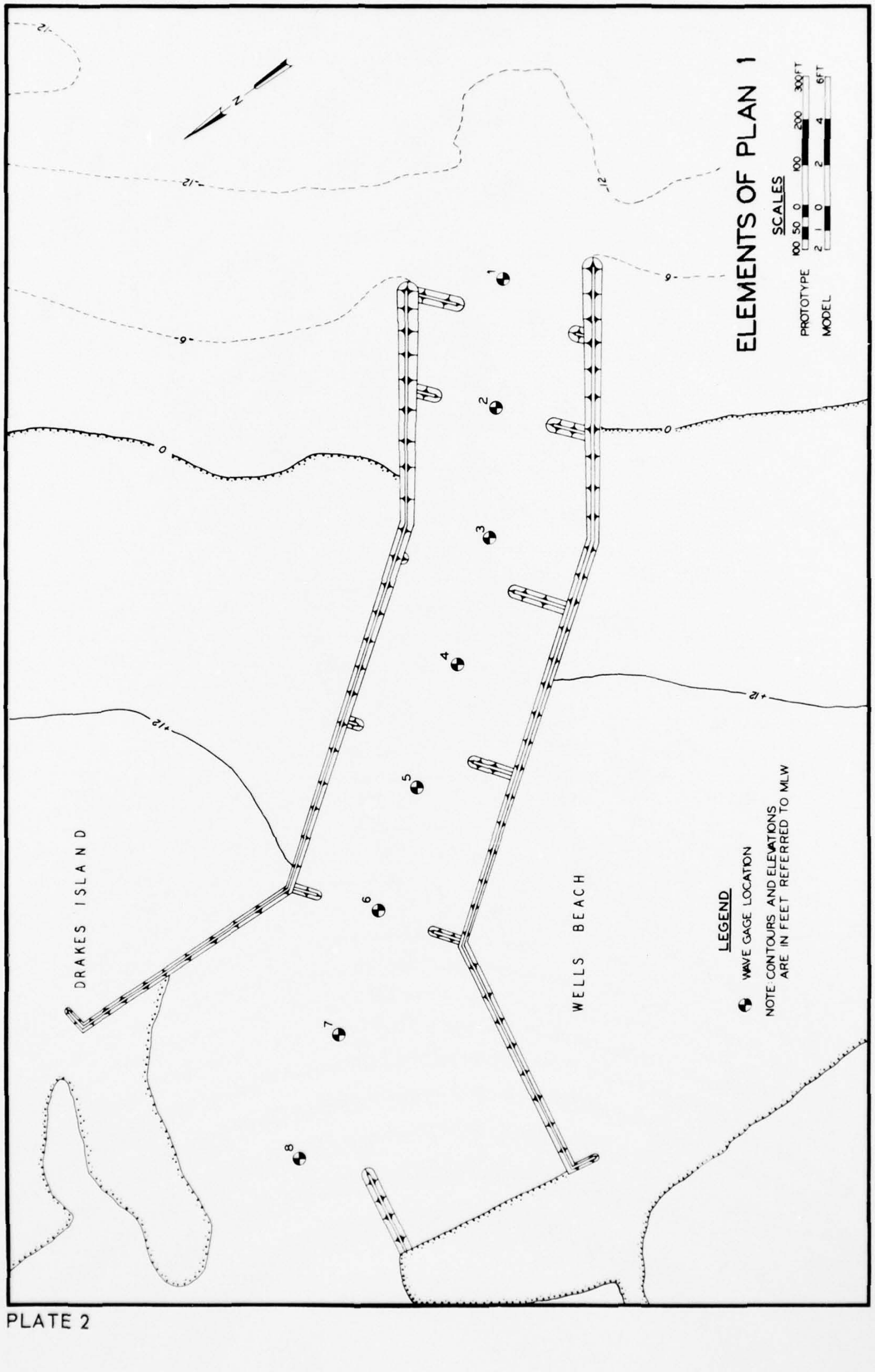
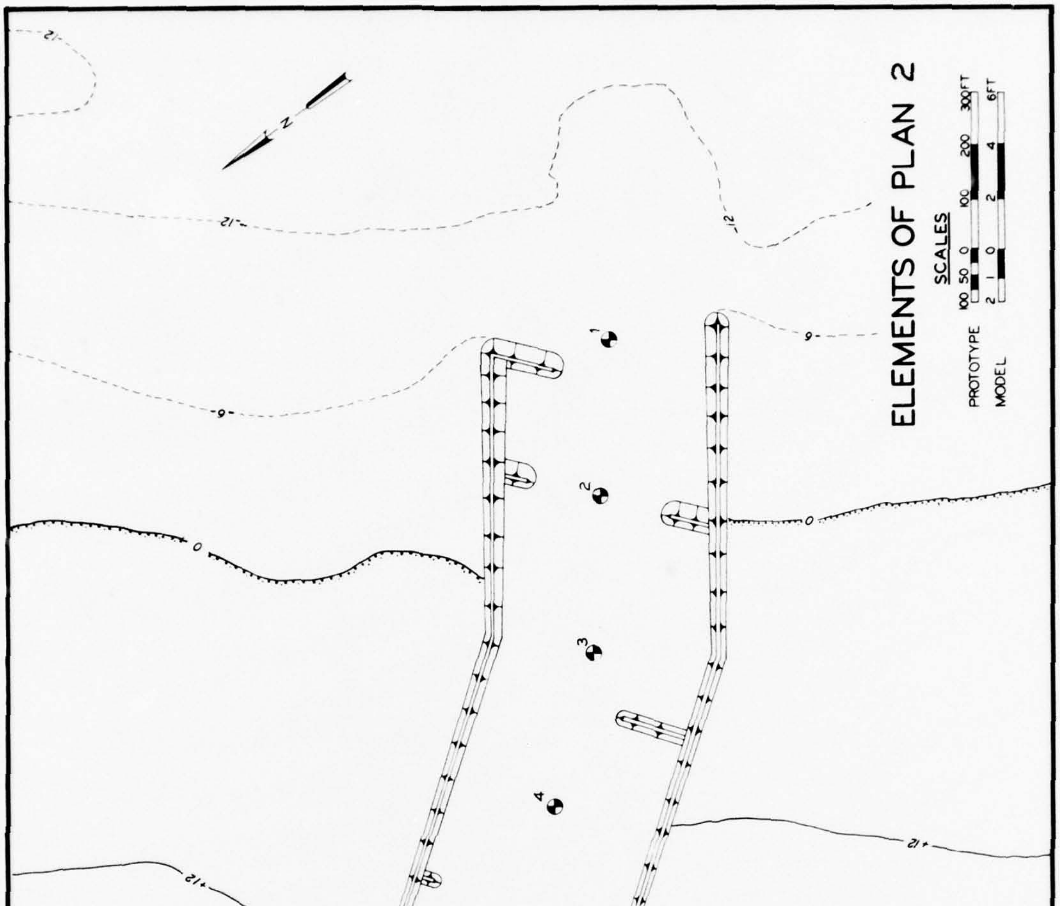
PROTOTYPE 100 50 0 100 200 300FT
MODEL 2 1 0 2 4 6FT
LEGEND
WAVE GAGE LOCATION
NOTE CONTOURS AND ELEVATIONS
ARE IN FEET REFERRED TO MLW
DRAKES ISLAND
WELLS BEACH
N
-12
-6
0
+12

PLATE 1



ELEMENTS OF PLAN 1
SCALES
PROTOTYPE 100 50 0 100 200 300 FT
MODEL 2 1 0 2 4 6 FT
LEGEND
WAVE GAGE LOCATION
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MLW
DRAKES ISLAND
WELLS BEACH
N
-12
-6
0
+12
1
2
3
4
5
6
7
8


ELEMENTS OF PLAN 2
SCALES
PROTOTYPE 100 50 0 100 200 300FT
MODEL 2 1 0 2 4 6FT
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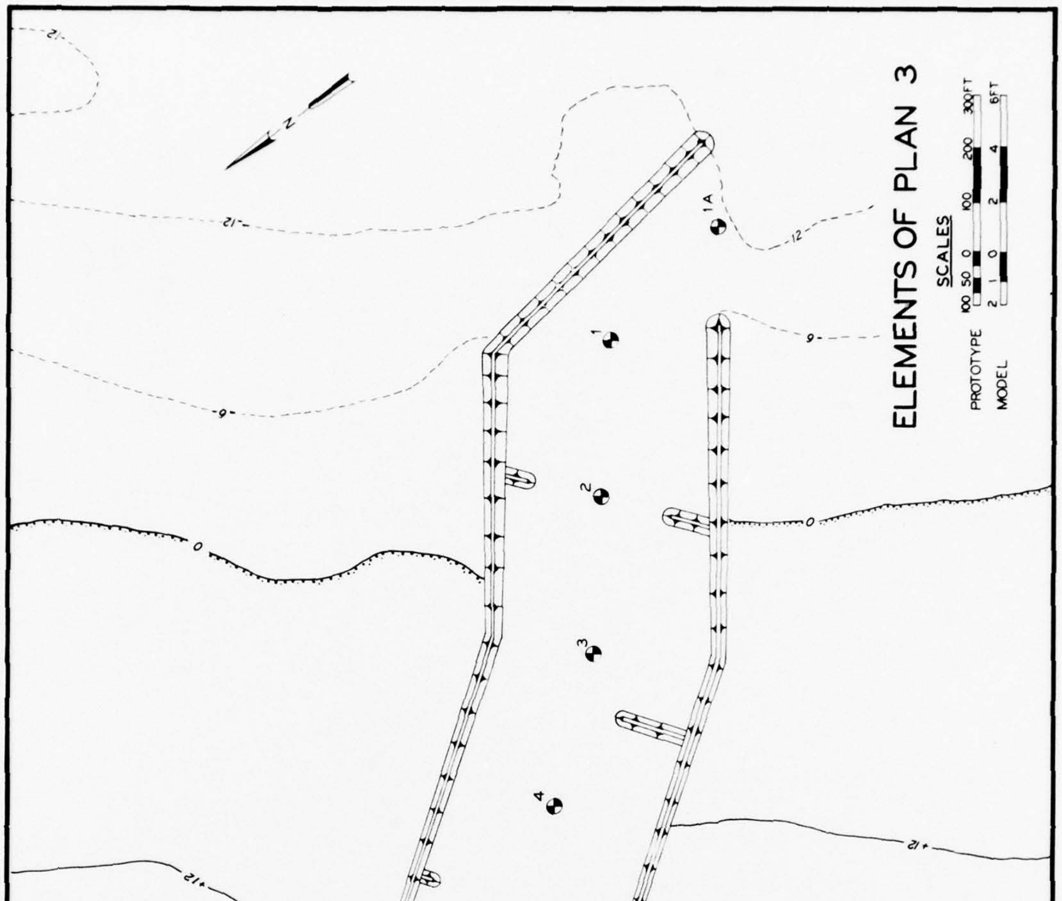
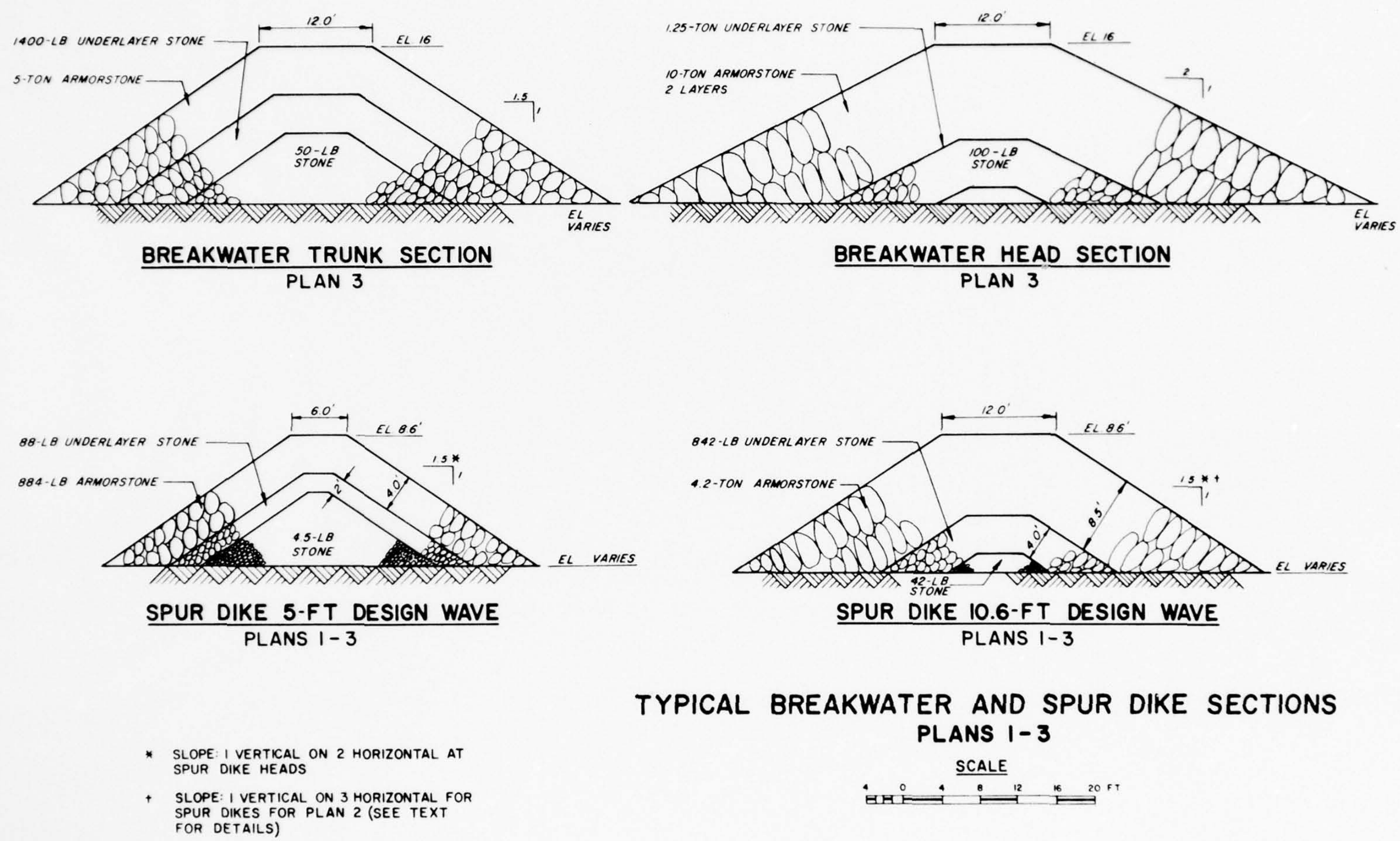
LEGEND
WAVE GAGE LOCATION
NOTE CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MLW
DRAKES ISLAND
WELLS BEACH



ELEMENTS OF PLAN 3
SCALES
PROTOTYPE 100 50 0 100 200 300FT
MODEL 2 1 0 2 4 6FT
LEGEND
WAVE GAGE LOCATION
NOTE CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MLW
DRAKES ISLAND
WELLS BEACH
N
1A
1
2
3
4
5
6
7
-12
-6
0
+12

PLATE 4



TYPICAL BREAKWATER AND SPUR DIKE SECTIONS
PLANS 1-3

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Bottin, Robert R

Design for harbor entrance improvements, Wells Harbor, Maine; hydraulic model investigation / by Robert R. Bottin, Jr. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1978.

24, [55] p., 5 leaves of plates : ill. ; 27 cm. (Technical report - U. S. Army Engineer Waterways Experiment Station ; H-78-18)

Prepared for U. S. Army Engineer Division, New England, Waltham, Massachusetts.

References: p. 24.

1. Harbor engineering. 2. Harbor facilities. 3. Harbor models. 4. Hydraulic models. 5. Water wave action. 6. Wells Harbor, Maine. I. United States. Army. Corps of Engineers. New England Division. II. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical report ; H-78-18.

TA7.W34 no.H-78-18